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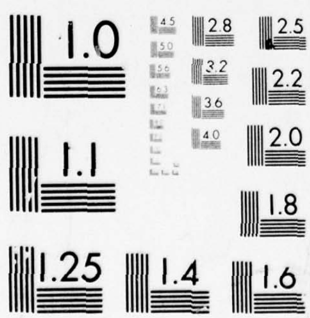


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# DREDGE DISPOSAL STUDY

# SAN FRANCISCO BAY AND ESTUARY





## APPENDIX M

## DREDGING TECHNOLOGY

SEPTEMBER 1975

6 DREDGE DISPOSAL STUDY, SAN FRANCISCO BAY AND ESTUARY.

APPENDIX M.

DREDGING TECHNOLOGY.

9 Final rept.

11 Sep~~tember~~ 1975

12 380 p.



U.S. Army Engineer District, San Francisco
Corps of Engineers
100 McAllister Street
San Francisco, California 94102

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# FOREWARD

In April 1972, the San Francisco District of the United States Army Corps of Engineers initiated a three and one-half year $3 million study to quantify the impact of dredging and dredged material disposal operations on the San Francisco Bay and Estuarine environment. The study is generating factual data, based on field and laboratory studies needed for the Federal, State and local regulatory agencies to evaluate present dredging policies and alternative disposal methods.

The study is set up to isolate the questions regarding the environmental impact of dredging operations and to provide answers at the earliest date. The study is organized to investigate (a) the factors associated with dredging and the present system of aquatic disposal in the Bay, (b) the condition of the pollutants (biogeochemical), (c) alternative disposal methods, and (d) dredging technology. The study elements are intended first, to identify the problems associated with dredging and disposal operations and, second, to address the identified problems in terms of mitigation and/or enhancement. The division into separate but inter-related study elements provides a greater degree of expertise and flexibility in the Study.

This report presents the findings of Appendix M, Dredging Technology. The overall study will be the basis for preparation of a composite Environmental Impact Statement for Dredging Activities in San Francisco Bay System. A draft final report on the entire study is scheduled for completion in December 1975.

The following is an index of appendices to be published in the Dredge Disposal Study:

| APPENDIX | REPORT | DATE PUBLISHED |
|---|---|---|
| | FINAL REPORT | |
| A | Main Ship Channel (San Francisco Bar) | June 1974 |
| B | Pollutant Distribution | |
| C | Water Column (Water Column-Oxygen Sag) | |
| D | Biological Community | August 1975 |
| E | Material Release | |
| F | Crystalline Matrix | July 1975 |
| G | Physical Impact | July 1975 |
| H | Pollutant Uptake | September 1975 |
| I | Pollutant Availability | |
| J | Land Disposal | October 1974 |
| K | Marsh Development | |
| L | Ocean Disposal | |
| M | Dredging Technology | September 1975 |

CONVERSION FACTORS

If conversion from the Metric to the British system is necessary, the following factors apply:

LENGTH

1 kilometer (km)=$10^3$ meters=0.621 statute miles=0.540 nautical miles
1 meter (m)=$10^2$ centimeters=39.4 inches=3.28 feet=1.09 yards=0.547 fathoms
1 centimeter (cm)=10 millimeters (mm)=0.394 inches=$10^4$ microns ($\mu$)
1 micron ($\mu$)=$10^{-3}$ millimeters=0.000394 inches

AREA

1 square centimeter ($cm^2$)=0.155 square inches
1 square meter ($m^2$)=10.7 square feet
1 square kilometer ($km^2$)=0.386 square statute miles=0.292 square nautical miles

VOLUME

1 cubic kilometer ($km^3$)=$10^9$ cubic meters=$10^{15}$ cubic centimeters=0.24 cubic statute miles
1 cubic meter ($m^3$) =$10^6$ cubic centimeters=$10^3$ liters=35.3 cubic feet=264 U.S. gallons=1.308 cubic yards
1 liter=$10^3$ cubic centimeters=1.06 quarts=0.264 U.S. gallons
1 cubic centimeter ($cm^3$)=0.061 cubic inches

MASS

1 metric ton=$10^6$ grams=2,205 pounds
1 kilogram (kg)=$10^3$ grams=2.205 pounds
1 gr (g)=0.035 ounce

SPEED

1 knot (nautical mile per hour)=1.15 statute miles per hour=0.51 meter per second
1 meter per second (m/sec)=2.24 statute miles per hour=1.94 knots
1 centimeter per second (cm/sec)=1.97 feet per second

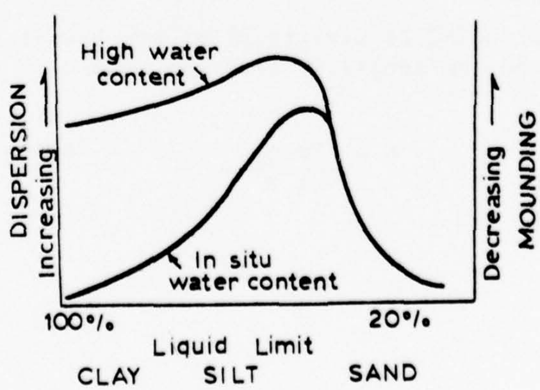
TEMPERATURE

Conversion Formulas $\quad °C = \dfrac{°F-32}{1.8} \qquad °F = 1.8(°C) + 32$

PREFACE

In order to define the type and degree of impact associated with aquatic sediment disposal, knowledge of the sediment release pattern generated by the dredging operation is necessary. Field work associated with other study elements gives an insight into the various types of release patterns. During studies on the San Francisco Bar (Main Ship Channel, Appendix A), the sandy sediments were found to react as discrete particles during disposal, depositing in a predictable pattern. The study showed a normally distributed deposition pattern with a maximum deposition of two inches directly beneath the hopper dredge. During the monitoring of oxygen depression associated with the disposal of Bay mud at Carquinez Strait (Water Column, Appendix C), an induced current of one knot perpendicular to the tidal current was observed at the bottom of the water column. Other monitoring at the Carquinez Strait site could account for complete transport of sediments from the site during a hopper release in fifteen minutes in the bottom three feet of the water column. During the monitoring of ocean disposal at a 100-fathom site (Ocean Disposal, Appendix L), a Bay mud with new construction clamshell dredging and barge transport, was observed to mound on the bottom in clumps. These and other studies led to a set of probable parameters which control the release pattern in the water column, the impact on the bottom and the degree of transport from the site. The primary objective of this study element was to evaluate these parameters in terms of the degree of influence on the pattern and to develop qualitative predictive relationships for water-sediment interaction in San Francisco Bay.

The changes in physical state of Bay sediments were investigated in the field during the dredging and transport phase. Two types of sediments were then taken to the laboratory for transport simulation and sediment release simulation. Although both sediments were classified as clays, there was a major difference in the properties of the sediment (difference in ratio of silt and clay). With cohesive sediments, the release pattern (degree of dispersion or mounding) can be correlated with the liquid limit and the moisture content of the sediment. The degree of dispersion or mounding depends on whether the sediment acts as a solid, a liquid or a transitional slurry. Cohesive sediments require a disturbance in terms of water added; whereas, sands, regardless of water content, act as a solid phase. This theorization is depicted below.



For qualitatively predicting the immediate release pattern in San Francisco Bay, a simple model using a small scale laboratory test on cloud growth combined with the expected moisture content due to the dredging method and operation, the depth of the disposal site and currents at the disposal site, should provide, within the variability of the prototype system, an evaluation of the loading of the water column, the degree of bottom impact and the rate of transport from the site. Care must be taken in scaling up the sediment volume. The volume must be evaluated in terms of the configuration and layout of hoppers or pockets of the barge. Additional objectives deal with questions on land disposal and marsh development, both of which were covered primarily through a literature review. This information will be applied to the results of other study elements to define the extent of environmental impact.

# DREDGING TECHNOLOGY STUDY
# SAN FRANCISCO BAY AND ESTUARY

by

JBF Scientific Corporation
2 Ray Avenue
Burlington, Massachusetts 01803

September 1975

Contract No. DACW 07-75-C-0045

Prepared for

U. S. Army Engineer District, San Francisco
Corps of Engineers
100 McAllister Street
San Francisco, California 94102

## ABSTRACT

A study was conducted to investigate dredging technology and to advance the state of knowledge regarding the short-term fate of dredged materials dumped from barges or hopper dredges. Particular attention was given to application of the study findings to protection of the aquatic environment in the San Francisco Bay Area.

Field tests evaluated physical and chemical properties of dredged materials in many conditions, such as in situ, and in the pockets of a hopper dredge and a barge. Physical properties including moisture content and strength parameters were found to be quite variable within the pockets. Vibrations while vessels are underway had no significant effect on material properties, but heeling and swaying of the vessel appeared to have some slight influence in decreasing the moisture content in the lower portions of material in a hopper dredge.

Laboratory simulations of bottom dumping barges and hopper dredges were conducted with silt and clay sediments from San Francisco Bay navigation channels. These tests were performed in glass walled tanks and evaluated by means of motion pictures and still photography of the simulated dumping operations. Behavior of the dumped material as a function of sediment type (silt or clay), water type (fresh or salt), vessel configuration (hopper dredge or barge), water depth, sediment percent moisture, and dumped volume was studied. Parameters observed in characterizing the data included descent velocity, cloud size, impact velocity, horizontal velocity following impact, and settling patterns. Moisture content appeared to be the primary variable determining behavior of dumped materials.

Engineering aspects of intertidal disposal for the purpose of marsh creation were evaluated. Control of fill elevations was found to be

enhanced by various means of predicting settlement and mechanical conditioning of deposited materials. Other considerations in marsh building were discussed, including tidal inlet design, means of excavating in marsh areas, and movement of salt through dredged material. This last subject, prompted by the desire to build marshes on abandoned salt evaporation ponds, was approached with a laboratory experiment. A lysimeter test was conducted to determine upward migration of sea salt through wet dredged material.

Certain aspects of land disposal were investigated with reference to conditions in the San Francisco Bay Area. Treatment processes for removing contaminants and for hastening drying were discussed and evaluated. Productive uses of dredged material were also considered.

## ACKNOWLEDGMENTS

The staff of the San Francisco District, Corps of Engineers was most cooperative and helpful in this project. John Sustar aided in formulating the scope of investigation, in arranging tests aboard dredging equipment, and in the process of steering the work, as it progressed, toward promising areas and away from those subjects which were seen to be fruitless early in the study. Paul Knudsen provided valuable background and references for the evaluation of marsh building. Particular appreciation is felt for the work of Corps divers Douglas Pirie and John Hendrick in gathering bottom samples under extremely adverse conditions of current, visibility, and temperature.

Woodward-Clyde Consultants of Oakland, California contributed to this study by performing geotechnical tests in the field and laboratory. Littleton Research and Engineering Corp. of Littleton, Massachusetts conducted the vibration measurements on the hopper dredge HARDING. Pacific Environmental Laboratory performed chemical analyses which required rapid evaluation and which therefore could not be performed after shipping to Burlington. Dumping tests in large tanks were performed at Alden Research, Inc., Holden, Massachusetts.

JBF Scientific Corporation staff contributing to the writing of this report were Edward Johanson, Stuart Bowen, Ronald Orner, and Jaret Johnson. George Henry and Stephen Greene conducted the dumping tests, and Mr. Greene also assisted in analyzing data from those tests. Raimond Bianchi was the photographer.

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## APPENDICES

# I. INTRODUCTION

This report documents the study results obtained on Contract DACW07-75-C-0045, conducted for the San Francisco District of the U.S. Army Corps of Engineers, as part of their three year program to investigate the environmental impact of dredging operations on San Francisco Bay.

The primary objective of the study was to evaluate types of dredges, methods of operation, and disposal in terms of modifying the physical reaction of San Francisco Bay sediments in the water column. Other objectives were to determine acceptable techniques for establishing a sediment base for marsh development in diked areas, and a review of the land disposal of polluted dredged material. Three work items were identified:

a. Evaluation of the dredging, transport, and aquatic disposal phases
b. Evaluation of intertidal disposal
c. Evaluation of land disposal

Each of these work items was accomplished as individual studies. The results are presented separately in the following chapters of this report.

The evaluation of the dredging, transport, and aquatic disposal phases was accomplished through a combination of literature review, analytical studies, field measurements and laboratory simulations. Field tests were conducted to obtain values of parameters of interest so that definitive laboratory simulations could be conducted. The investigation concentrated on examining the physical characteristics and oxygen demand of in situ and dredged material. The intent was to examine changes in the characteristics of the material from in situ, through the dredge, and during the transport phase. The changes were then related to the dispersion that occurs when the material is bottom

dumped. Numerous motion pictures and photographs were taken to be used for quantitative analysis of the phenomena and to document the various parts of the program.

The evaluation of intertidal disposal was conducted through visits to potential marsh areas in the Bay area and a comprehensive literature review. A laboratory experiment was also conducted to study the movement of salt through Bay dredged material.

The evaluation of land disposal was conducted through a literature review supplemented with measurements taken inside and outside of a land disposal area on Mare Island.

In the following sections each chapter addresses a separate task. The appendices contain supplementary data for Chapter III.

## II. DREDGING EQUIPMENT USED IN THE BAY AREA

Most of the dredging in the major navigation channels of San Francisco Bay is performed by hopper dredges or clamshell dredges. Operations with each of these types of equipment were monitored in the field during this study.

### A. Hopper Dredges

San Francisco Bay is serviced by the BIDDLE and the HARDING hopper dredges.

The BIDDLE, a side-drag hopper dredge built in 1947, is the largest of the hopper dredges assigned to the Portland District, Corps of Engineers. The overall length is 352 feet 8-3/4 inches with a beam of 60 feet. Maximum and minimum dredging depths are 70 and 22 feet, respectively. Top speed is 14.8 statute miles per hour and personnel complement is 15 officers and 77 men. Dredging is accomplished with two 1,150 horsepower pumps. The suction line is 30 inches in diameter and the discharge line is 28 inches. Normal hopper capacity is 3,060 cubic yards, but added equipment has reduced the actual operation capacity to about 2,900 cubic yards.

Field testing in this study was conducted on the hopper dredge HARDING (Figure 2-1). The HARDING, built in 1939, has an overall length of 308 feet 2 inches with an overall beam of 73 feet. It has an assigned crew of 12 officers and 53 men. Dredging is accomplished with two 1000 horsepower pumps. Each pump has a 22 inch suction line and a 20 inch discharge. The maximum drag depth is 62 feet. The two pumps deliver an average of 208 cubic yards of material and water slurry per minute to the hoppers. The hoppers will hold 2682 cubic yards without sideboards and 2768 with sideboards. Typical

dredging speed is 1.8 knots and transiting speed is 8-10 knots.

B. Clamshell Dredges

Mechanical dredging with clamshell dredges is performed by contractors in the Bay Area. The BOSTON, (Figure 2-2), operated by Great Lakes Dredge and Dock Co., was working in the Alameda area during the time of this study, and that operation was the subject of a portion of the study's field work.

During the dredging operation, a bottom dumping barge was tied alongside the BOSTON and filled with the clamshell. An 18 cu. yd. bucket was used and a fill and dump cycle took about 1 min. The bucket opened to a dimension of 20 feet and was 7 feet wide. The bucket was several feet deep and there was direct access to the dredged sediment from the top of the bucket.

In practice, the bucket is dropped and sinks into the sediment. The dredge carriage is then slewed causing the bucket to dig into the sediment. As it is raised, very little material escapes from the bottom but water pours from the top surface. The material in the bucket during the Alameda operation had the consistency of toothpaste. While the material may be disturbed, the water most probably only scours the top surface of the material in the bucket.

These dredges, the HARDING and the BOSTON, typify the equipment used for much of the dredging activity in the Bay Area. The field observations made on these vessels are described in Chapter III.

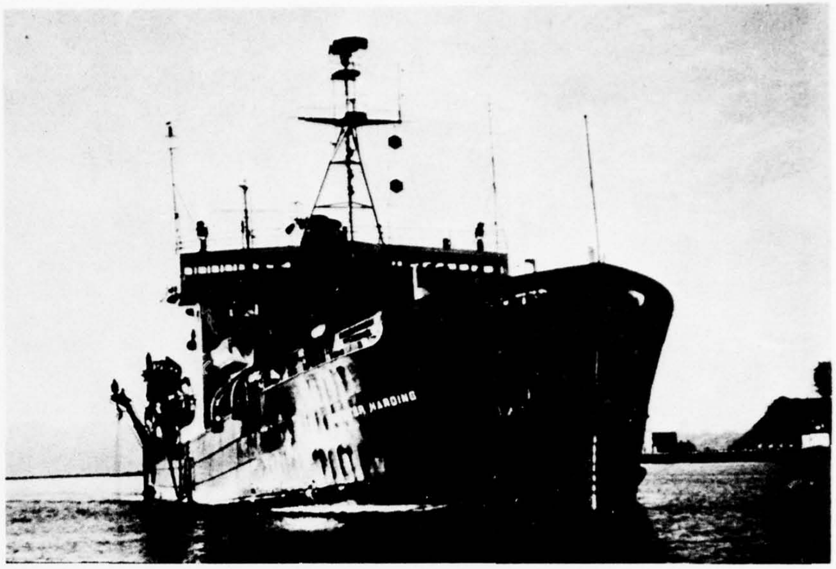

Figure 2-1. The hopper dredge HARDING.

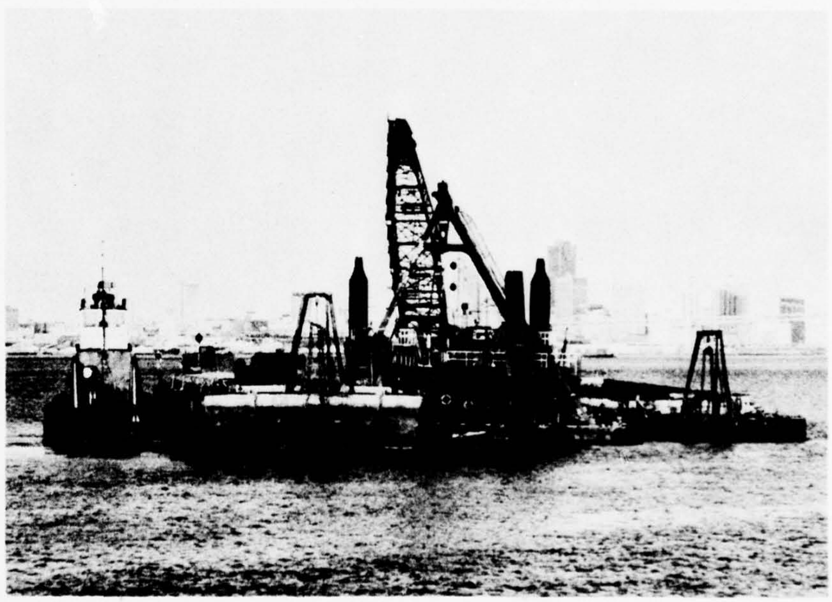

Figure 2-2. The vessels used in the clamshell operation in this study. Left to right: tow boat, barge, the clamshell dredge BOSTON, and another barge.

## III. EVALUATION OF DREDGING, TRANSPORT, AND AQUATIC DISPOSAL PHASES

The approach that was adopted to achieve the goals of the dredging, transport, and disposal phase consisted of a field measurement program and a laboratory simulation program in support of analytical studies. The results of the field and laboratory programs will be presented in the following sections under Dredging Phase, Transport Phase, and Disposal Phase. Representative data is tabulated in Appendix A.

### A. Field Measurements Program

#### 1. General Description

During the period of March 11th through March 20, 1975, field tests were conducted to:

- obtain data on the change in *in situ* physical characteristics of sediments caused by dredging with a hopper dredge and a clamshell dredge
- obtain data on the sediment oxygen demand (IOD and COD) of *in situ* sediments and dredged material
- characterize the change in dredged material caused by the transportation process in a hopper dredge

The measurements were made in conjunction with the Clamshell Dredge BOSTON at the Alameda Naval Air Station and the Hopper Dredge HARDING in Mare Island Straits. Sampling and measurements of IOD and penetration (2 inch lead sinking sphere) were made by JBF Scientific Corporation (JBF). Woodward Clyde Consultants (WCC), under contract to JBF, made measurements of *in situ* shear strength and penetration resistance. COD measurements were made, for JBF, on samples of *in situ* and dredged material by Pacific Environmental Laboratories (PEL).

The BOSTON was working the Alameda Naval Air Station entrance and pier area, in the following locations:

Location 1: 150 feet SW of Buoy 5

Location 2: 500 feet SE of Pier 3

Location 3: 100 feet inside channel, half way between Buoys 3 and 5.

The HARDING was dredging in Mare Island Straits and dumping in Carquinez Straits. The cycle was short, allowing about 15 minutes from time dredging ceases until dumping took place.

Table 3-1 shows the type of measurements made on the BOSTON clamshell dredge and its barges, and Table 3-2 shows the measurements made on the HARDING hopper dredge.

Table 3-1 BOSTON Measurements

| Parameter | Methodology |
|---|---|
| Shear Strength | In clamshell bucket, sampler, and barge pockets, using field torque vane |
| Penetration | In barge, using flat plates and 2 inch lead shpere |
| Dissolved Oxygen | In barge water, turbidity plume, and background using DO meter |
| Oxygen Demand | From samples taken in barge and in bucket (clamshell). IOD measured immediately and COD on iced samples. |
| Density, Percent moisture, and Grain Size | Measured in laboratory from samples taken on barge and in bucket. |

Table 3-2

HARDING Measurements

| Parameter | Methodology |
|---|---|
| Shear Strength | In samples from the chute and the hopper, and in the hopper, using a field torque vane |
| Penetration | In the hopper, using flat plates and a 2 inch diameter lead sphere |
| Dissolved Oxygen | In the hopper water, using a DO meter |
| Oxygen demand | From samples taken in the hoppers, and hopper chutes. IOD measured immediately and COD on iced samples |
| Density Percent Moisture and Grain Size | Measured in laboratory from samples taken in the chutes and hoppers |
| Ship Vibrations | Measured on the walls of the hoppers, and directly in the hoppers, using accelerometers and velocity pickups |

Table 3-3 shows the type of measurements made on _in situ_ samples obtained by divers.

Table 3-3

_In Situ_ Measurements

| Parameter | Methodology |
|---|---|
| Shear Strength | In relatively undisturbed samples obtained by Scuba Divers |
| Oxygen Demand | IOD measurements (immediately) and COD measurements (on iced samples) |
| Density Percent Moisture and Grain Size | Measured in laboratory |

## 2. Measurements

### Shear Strength Measurements

*In situ* shear strength of dredged materials aboard the BOSTON and the HARDING were determined using a four-bladed, rectangular field vane with 3 inch by 6 inch dimensions. The vane was lowered into the mud, attached to E-size drilling rods which were clamped into a vertical position through a portable platform attached to ship's railings. Torque was applied at a rate of approximately 15 degrees per minute at the top of the E-rods with a torque wrench which indicated inch-pounds of torque. The tests were carried out generally in accordance with ASTM D-2573-72 "Standard Method for Field Vane Shear Test on Cohesive Soil." However, due to time limitation aboard the respective vessels, the torque for the tests was applied at a rate slightly higher than the ASTM standard, i. e., at approximately 0.25 degrees/second instead of 0.1 degrees/second. Several tests were run aboard both vessels during which the torque needed to turn the E-rods without the field vane was recorded. The information recorded from those tests was subsequently used to calculate a shear strength correction factor to allow for rod friction.

Figure 3-1 shows the vane after withdrawing it from the barge pocket and Figure 3-2 shows the vane measurements being made in the hopper of the HARDING.

### Penetration Tests

A circular steel plate of 6 inch diameter and a square steel plate of 12 inch sides were used to measure pene-

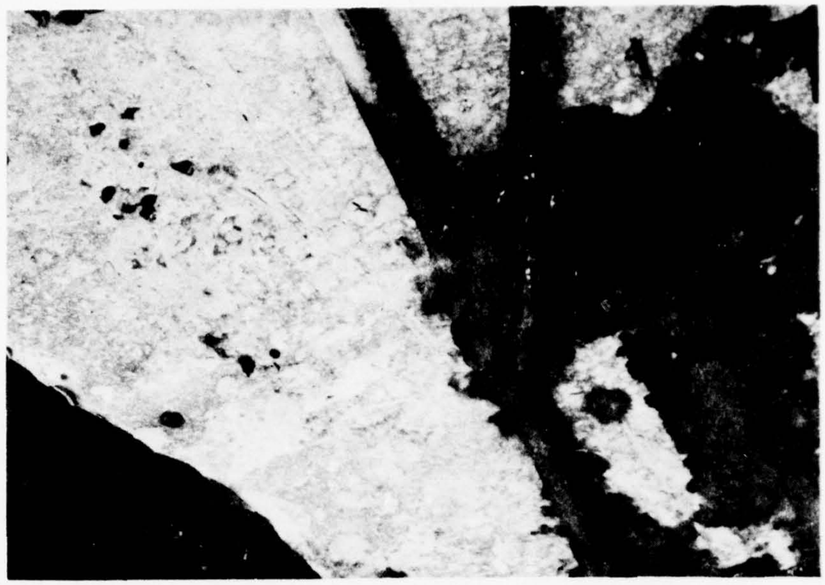

Figure 3-2. Making Torque Vane Measurements on the Hopper Dredge

tration resistance of the dredged materials in the respective vessels. The plates were attached to E-size drilling rods and lowered into the hoppers or barge pockets. Settlement, with respect to time, was recorded for each increment of weight applied, as was the depth of the plate at each increment. Figures 3-3 and 3-4 show penetration tests being conducted on a barge.

### Sinking Sphere Tests

In order to further delineate penetration resistance and stratification in the various materials, and to attempt to calculate the bearing capacity of the materials, a 2 inch diameter lead sphere was lowered into the hoppers and/or barge pockets, and its settlement was recorded with respect to time. The depths of resistance to the sinking of the sphere were also recorded as were the depths at which the sphere apparently came to rest.

### Oxygen Demand

Measurements of IOD, COD, and dissolved oxygen were made in a number of locations using a method modified from Standard Methods [1] for IOD, and Standard Methods and the EPA sediment manual [2] for COD. Dissolved oxygen was measured either *in situ* or immediately upon collection using a DO meter.

### Physical Characteristics of Dredged Materials

In order to determine basic physical characteristics of the dredged materials, numerous samples were obtained at various locations, depths and times using several methods. Some samples were recovered with a 50cc syringe attached

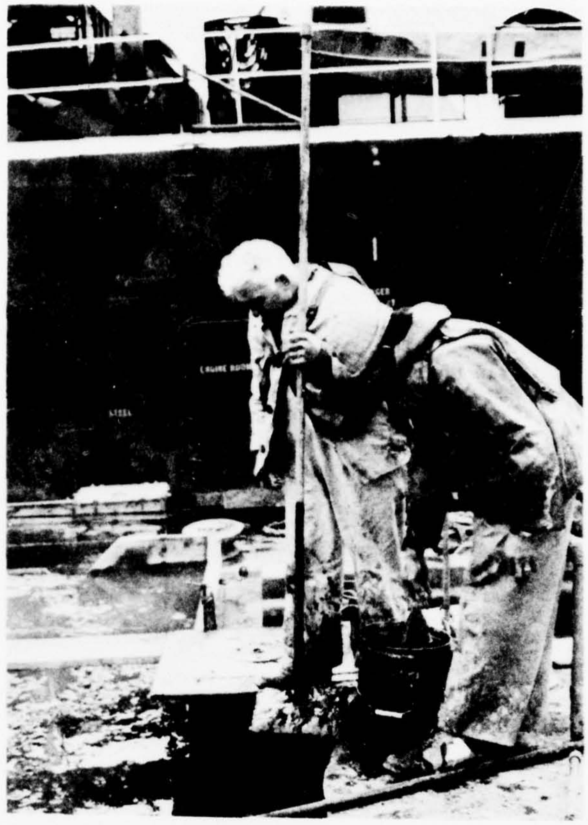

Figure 3-3. Penetration Tests Using a 1 sq ft Plate

Figure 3-4. Penetration Tests in Pocket of Barge

to a drill rod and operated from the ship's deck by pulling on a string attached to the syringe plunger. When the syringe was recovered the sampled materials were squeexed through a 1/4-inch opening at the head of the syringe into plastic containers. Other samples were obtained using a coring tool (BU Core Samples) and a grab sampler (grab-samples). Again the samples were transferred from the sampling devices into plastic containers. Figure 3-5 shows the syringe sampler used.

In order to obtain in situ samples of the materials on the bay bottom, scuba divers recovered samples by pushing PVC tubing and/or brass tubes ranging in diameter from 2-1/2 to 6 inches into the bay sediments and capping both ends of each tube prior to retrieval. Figure 3-6 shows a typical sampler.

All samples were carefully sealed in the field in order to preserve the natural moisture content of the soils, and selected samples were taken to the laboratory for examination and testing. Tests for moisture content were carried out in accordance with ASTM D-2216-71 "Laboratory Determination of Moisture Content of Soil." Dry densities were calculated based on wet weight, volume and moisture content of samples.

Grain size determinations consisting of hydrometer tests were made of selected samples from both dredges in accordance with ASTM D-442-63 (Reapproved 1972). Two hydrometer tests were run on each sample tested. One was run exactly following the ASTM procedure, which calls for the addition of a dispersing agent to the fluid in the sedimentation cylinder. The purpose of this dispersing agent is to counter-

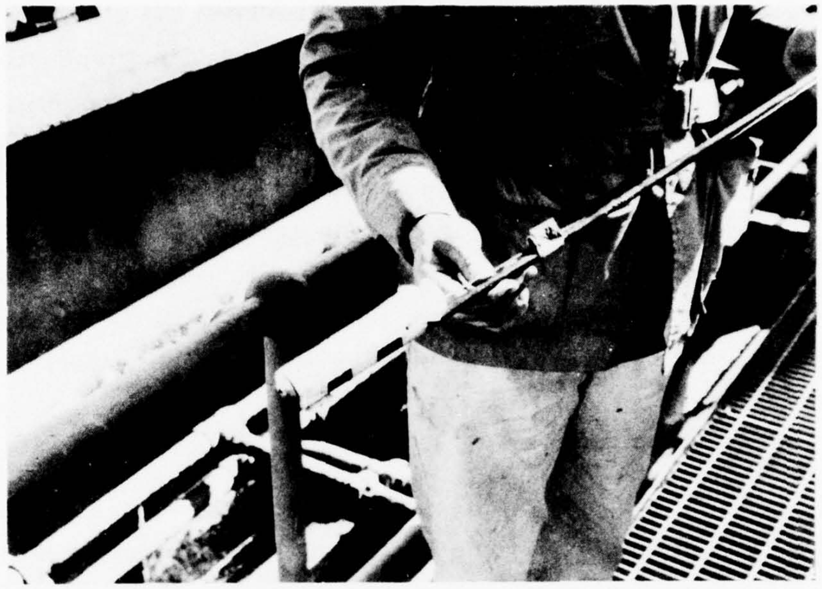

Figure 3-5. Syringe Coring Device

Figure 3-6. In Situ PVC Sampler

act the chemical attraction between the clay-size particles in the sample, causing them to stay in suspension and thus allowing an accurate determination of the percentage of clay sizes in a given sample. The second hydrometer test was run following the same ASTM procedures, but without the addition of a dispersing agent. The purpose of the non-dispersed tests was to attempt to recreate the field condition in dredges where any sedimentation of the materials would occur without a dispersing agent in the water.

Vibrations

Vibration measurements were made on the HARDING to obtain data for the design of the laboratory haul simulation. Harmonic motions were recorded for the forward section of the Number 3 hopper using a special vertical "mud" transducer designed to be inserted directly into the dredged material. Vibration measurements were also made of the hopper structure adjacent to the mud probe. Figure 3-7 shows the mud probe being installed in a hopper of the HARDING.

## B. Laboratory Simulation Program

### 1. General Description

While dredging has been carried out for many years, scientific investigations related to it are a relatively new activity. Since full scale measurements are extremely expensive to conduct, and the results obtained can be highly variable, laboratory studies were conducted to identify the effect of changes in physical characteristics of the dredged material on the transportation and disposal phases. The penalty

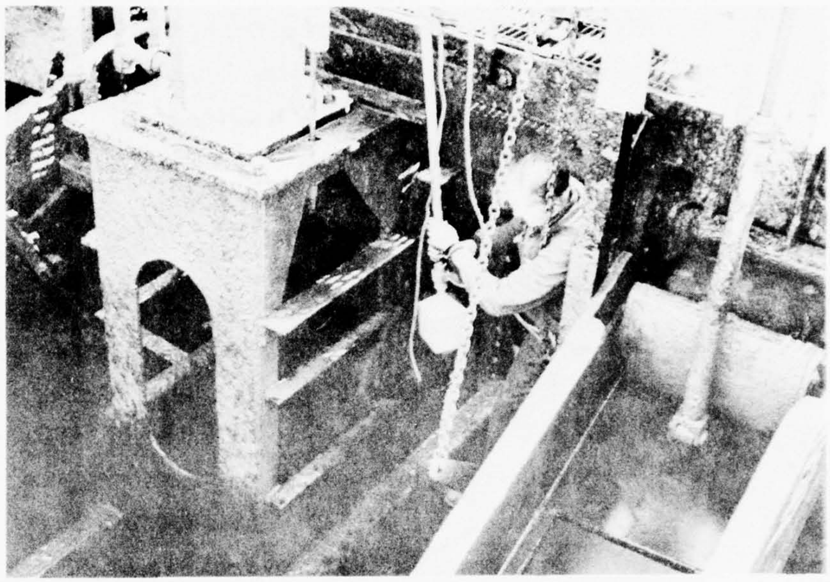

Figure 3-7. Mud Transducer Being Installed in the Hopper

for enhanced control in the laboratory environment is uncertainty introduced by scaling of the operations.

A set of controlled experiments were designed to establish the effects, if any, of the transportation and dumping operations on the ultimate dispersion of the material. The contract scope of work states: "Qualitative predictive conditions shall be extrapolated to actual transport modes and distance and disposal site conditions." Based on a carefully designed set of experiments and the use of mathematical models, an attempt has been made to be more quantitative than was requested in the RFP.

2. Transportation Phase

This portion of the laboratory simulation program, which was performed by Woodward-Clyde Consultants, simulated the transportation phase of the dredging cycle. The program was set up to determine the time and vibration effects on the dredged materials while in transit between the dredging and disposal sites. The samples used in the program were reconstituted at several moisture contents. Two distinct materials were used. These materials were recovered as bulk samples in 55 gallon steel drums which were dragged across the bay bottom. Twelve of these barrels, six "silt" barrels and six "clay" barrels were sent to the WCC soil laboratory where the testing program simulating the transport or haul phase was carried out. The "silt" material came from Pinole Shoals and the "clay" material from Mare Island Straits.

The tests on materials from the bay mud barrels consisted of index and classification tests, laboratory vane shear

tests and the Modular Test System (MTS) tests. Index and classification tests were run to identify the types of soils obtained and to allow the reconstitution of generally homogeneous bulk samples for use in the vibration tests.

Laboratory vane shear tests were run at three depths in each barrel to measure the shear strength of the materials as received. Tests were also run several times during a 24-hour period on random samples in separate cylinders in order to determine any thixotropic effects on the muds. Thixotropy is the property of a material that enables it to stiffen in a relatively short time on standing, but on agitation or manipulation to change to a very soft consistency or to a fluid of high viscosity. The process is completely reversible. The tests exceeded the time period needed to simulate the haul distance.

The vibration test program was run on the Modular Test System (MTS) equipment, which is generally used for cyclic or dynamic testing of soils. This specialized equipment allows samples to be vibrated at a wide range of amplitudes and frequencies.

Sediment columns of the two materials were tested in two foot high, clear plastic cylinders at two frequencies and amplitudes and at four water contents. The two frequencies covered the range of frequencies as measured in the field. The four water contents covered the range that is typical for the water contents of the dredged materials in a hydraulic pipeline dredge, hopper dredge, and clamshell dredge. The time period of each test simulated a haul distance of 60 miles at a speed of 9 knots.

Each test was duplicated in a static mode, i.e., while one sediment column was vibrated in the MTS frame, an identical column was left undisturbed for the same time period with the purpose of comparing time effects with vibration effects. An identical set of samples was recovered from three depths in each cylinder, before and after each test, for water content and dry density determinations.

The amount of sedimentation, i.e., the depth to the water/solids interface in each cylinder after each test, was recorded. Laboratory vane shear strength measurements were made for the lower water content tests, but the readings were so low (within the accuracy of the instrument) that the shear strengths were considered zero.

## C. Dredging Phase

### 1. General

Dredging in San Francisco Bay is primarily accomplished by the U.S. Army Corps of Engineers using hopper dredges, such as the BIDDLE and the HARDING, or by contractors using clamshell dredges. The U.S. Navy also dredges in the Mare Island area using a hydraulic pipeline dredge.

Without regard to the dredging method used, the dredge selectively removes in-place material from some prescribed area of the Bay, and disposes of it by placing it into a barge or hopper, placing it in a landfill area, or returning it to the water at a nearby location. All of the dredging operations involve:

- the dredge selectively removing the in-place material

- near-field effects during the dredging operation
- changes in the physical characteristics of the in-place sediments

These are the three considerations that will be addressed in the Dredging Phase of this report.

2. Dredge Control Capability

The capability of controlling the dredge to allow selective removal of sediments is important from a number of points of view, because minimizing the volume of material removed minimizes the environmental effects in both the dredging and disposal areas, reduces the amount of sediment requiring disposal, and reduces the project costs.

For purposes of this report is is convenient to separate the discussion of hopper dredges and other dredges (clamshell and pipeline).

Hopper Dredges

The inherently good control capability of hopper dredges is due to their twin screw, electric drive propellors that provide stable speed control all the way down to zero rpm in both forward and reverse directions. The ships normally can turn on their yaw axes. The rudders are unusually large to insure positive steering at low water speeds.

There are two components of dredge locations to be concerned with: the surface location of the dredge relative to the desired in-place sediment and the vertical location of the drag arm relative to the desired project depth.

In a normal operation, the hopper dredge lines up on predetermined marks, or ranges, and runs down a channel dragging arms on either side. The dredged material is placed in hoppers, and when the hoppers are full the material is transported to the dump site and bottom dumped.

While it is possible to provide the hopper dredge with a precision navigation system, this is not usually done. Material dredged with the dragarms is from two strips on the bottom approximately separated by the beam distance of the vessel. Each strip is perhaps 10 feet wide. The nature of the sediment-water interface in the Bay is such that surrounding material may immediately flow into these dredged strips and repeated runs are necessary, each slightly increasing the channel depth.

While an experienced operator can quite precisely run a predetermined range, the hopper dredge is most effective in channels that allow the dredge to remove material over a substantial distance via repeated runs. It is not effective in selectively removing small regions of material, or in narrow channels that restrict its maneuverability.

The second component of positioning to consider is the vertical extent which is determined by the depth that the dragarm operators allow the dragarms to sink into the sediment This is determined by their judgement of the density of flow coming into the hoppers.

Figure 3-8 shows a record of the vertical draghead depth control on the HARDING hopper dredge operating in Mare Island Straits. The starboard draghead operator was operating at a depth substantially greater than the port operator and his depth changes were of a larger magnitude. While some of this difference may have been due to the shape of the channel being dredged, the inherent subjectivity of the operator is undoubtedly a factor.

Thus, the requirements for maneuvering the dredge, the

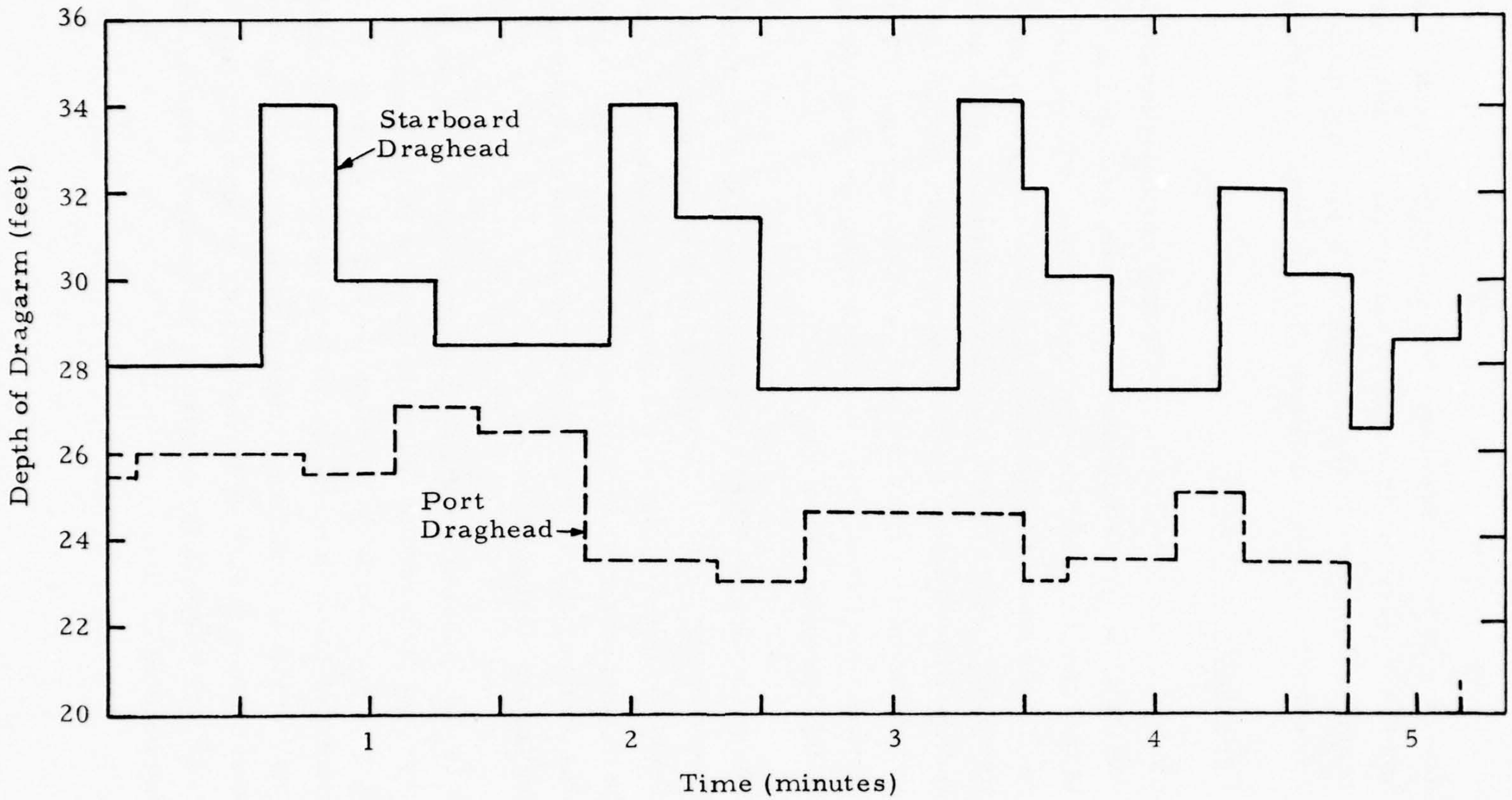


Figure 3-8. Vertical Draghead Depth Control on Hopper Dredge - HARDING
Simultaneous Readings for Both Dragheads

dredging of strips separated 60-70 feet from each other, and the subjectivity inherent in the dragarm depth control, considerably restrict the ability of a hopper dredge to selectively remove small quantities of in-place sediments.

Clamshell and Pipeline Dredges

Control of motion with these dredges normally involves "walking" on spuds with the thrust being provided by reeling in line that has been attached to anchors. The dredge is positioned on a line with the aid of tugs and it then drops its spuds. Anchors are placed well forward of the dredge and the dredge walks forward by reeling in anchor line and swinging on either the port or starboard spud. This motion allows the dredge to slowly crab, or yaw, forward along a predetermined line.

The predetermined lines are often established by installing lasers on the beach. The dredge Captain positions his dredge relative to the visual laser and channel marks that have been installed by a pre-dredge survey team. Using this method, the dredge may be positioned to within several feet of any desired location.

It is also possible to utilize the navigation techniques of pre- and post-dredge survey techniques if additional accuracy is required. Tables 3-4 and 3-5 present a summary of these methods. It is important to recognize that the visual positioning method using a laser is subject to serious degradation during periods of poor visibility (i.e. fog) whereas the electronic positioning systems are independent of this.

TABLE 3-4

Comparison of Distance Measuring Techniques

| Energy | Detection Technique | Velocity Limit | Range | Resolution | Accuracy |
|---|---|---|---|---|---|
| Radio | | | | | |
| LF | Phase | NS* | 1000 miles | 1 mile | |
| MF | Pulse | NS | 100 miles (not line-of-sight) | 300 ft | |
| HF | Phase | NS | 100 miles (not line-of-sight) | 3 ft | |
| μW | Pulse | NS | 100 miles (line-of-sight) | 0.1 meter | |
| μW | Phase | NS | 100 miles (line-of-sight) | 0.1 meter | ± 1 meter |
| μW | Doppler | NS | Unlimited | 0.1 ft/sec | ± 0.1% of distance traveled |
| Optical | Pulse | NS | 40 miles | 1 ft | |
| | Phase (Modulation) | NS | 2 miles | 0.01 ft | |
| | Phase (Coherent) | NS | 200 ft | 1 μ inch | |
| | Doppler | NS | Unlimited | 0.001 ft/sec | |
| Acoustic | Pulse | 10 mph | 12 miles | 1 ft | |
| | Phase | | 12 miles | 1 ft | |
| | Doppler | 100 mph | Unlimited | 0.1 ft | ± 0.1% of distance traveled |
| Inertial | Acceleration | Unlimited | Unlimited | 0.01 ft/sec$^2$ | ± 1 mile/hr of time traveled |

* NS - Not significant.

TABLE 3-5

Distance Measuring Equipment for Static Applications

| Company | Model | Range | Energy | Resolution | Accuracy | Weight | Cost, $ |
|---|---|---|---|---|---|---|---|
| Spectra Physics | Geodolite 3G | 40 miles | Optical | 1 mm | ±2 mm + 1 ppm | 150 lb | ≈50,000 |
| K&E (Laser Systems) | Microranger | 3 km | Optical | 1 mm | ±5 mm + 2 ppm | 13 lb | 4,000 |
| | Ranger II | 4 miles | Optical | 1 mm | ±5 mm + 2 ppm | 36 lb | 8,200 |
| | Ranger III | 8 miles | Optical | 1 mm | ±5 mm + 2 ppm | 36 lb | 8,700 |
| | Rangemaster | 40 miles | Optical | 1 mm | ±5 mm + 1 ppm | 55 lb | 18,000 |
| AGA | Geodometer - 6B | 20 km | Optical | | ±5 mm + 1 ppm | 24 kg | 7,300 |
| | Geodometer - 8 | 50 km | Optical | | ±6 mm + 1 ppm | 36 lb | 16,000 |
| | Geodometer - 700 | 3 miles | Optical | | | 62 lb | 15,000 |
| | Model 76 | 3 km | | 1 mm | ±10 mm + 1 ppm | 20 lb | 4,100 |
| Zeiss | SM-11 | 2 km | Optical | | ±10 mm | 36 lb | 13,500 |
| | Reg Elta 14 | 2 km | Optical | | ±10 mm | 36 lb | 27,000 |
| Wild | DI-10 Distomat | 2 km | Optical | | ±10 mm | | 6,200 |
| | DI-150 Distomat | | Microwave | | | | |
| Cubic | Cubitape | 2 km | Optical | | ±5 mm + 10 ppm | 12 lb | 4,000 |
| | Electrotape | 50 km | Microwave | | ±10 mm + 3 ppm | | 5,700 |
| Hewlett Packard | 3800 B | 2 km | Optical | 2 mm | ±5 mm + 7 ppm | | 4,200 |
| Tellurometer | MA-100 | 2 km | Optical | | ±2 mm + 1 ppm | 30 lb | |
| | CA-1000 | 6 miles | Microwave | | | 7 lb | 4,000 |
| | MRA-3 | 50 km | Microwave | | ±15 mm + 3 ppm | 40 lb | |
| | MRA-4 | 50 km | Microwave | | ±3 mm + 3 ppm | 40 lb | 9,000 |
| | MRA-101 | 65 km | Microwave | | ±20 mm + 3 ppm | | 3,500 |
| Scintrex | Akkuranger Mk II | 1 km | Optical | 3 mm | ±10 mm + 10 ppm | | |
| Sokkisha | SDM-3 | 1 km | Optical | | ±10 mm | 40 lb | |
| aus Jena | EOK-2000 | 2 km | Optical | | ±10 mm | | |
| Applied Devices | Micro-Surveyor Model 99 | 50 km | Microwave | | ±15 mm + 4 ppm | 112 lb | |

The second positioning consideration is the ability to selectively place the bucket, or the cutter head, directly on the sediment to be removed. Since the location of the dredge can be established to within several feet, and the bucket and ladder geometrics are known, these dredges have the capability of selectively removing sediment anywhere in the bay from a spot as small as perhaps 30-50 square feet. In normal operation this might be a substantially larger area since it is unlikely that the time will be taken to locate the dredge to these accuracies. However, the capability to selectively remove in-place sediments is far superior using a clamshell, or pipeline dredge, than with a hopper dredge, in both horizontal and vertical extent. This is especially important when considering highly polluted sediments.

## 3. Near-field Effects

While a dredge is operating, a number of near-field effects are of interest, such as turbidity generation and oxygen depletion. Little data exists on these effects.

### Turbidity

Near-field effects are generated by the hopper dredge due to:

- the action of the dragheads and propeller wash
- wier overflow

The dragheads are mounted on the ends of large diameter pipelines (20-30 inches) and towed through the sediment that is to be dredged. While a search of the literature indicates that direct observations have never been made, it can be assumed that the dragheads disturb the bottom and resuspend material. The amount of material resuspended is most probably quite small, at least in areas such as Mare Island Straits and the channel entering the Alameda Naval Air Station. In both of these locations divers experienced difficulty in locating a "bottom" and found a density gradient that extended over a depth of more than 5 feet. This variable density layer is described in more detail in a following section. It appears that the dragarm moves horizontally through a region with a large vertical gradient in density and viscosity. Under these conditions, and considering the rate at which material is being taken into the pipeline, the disturbance should be small.

Wakeman, et al [3] have reported turbidity levels downstream of a hopper dredge, as shown in Figure 3-9. The data clearly show elevated levels of turbidity throughout the water column, with the highest levels being near the bottom. This component is generated by a combination of the dragheads and the twin screws and it is not possible, at this time, to quantify the effect due to these two parameters separately. It is interesting to note that the plume

TRAILING SUCTION HOPPER DREDGE

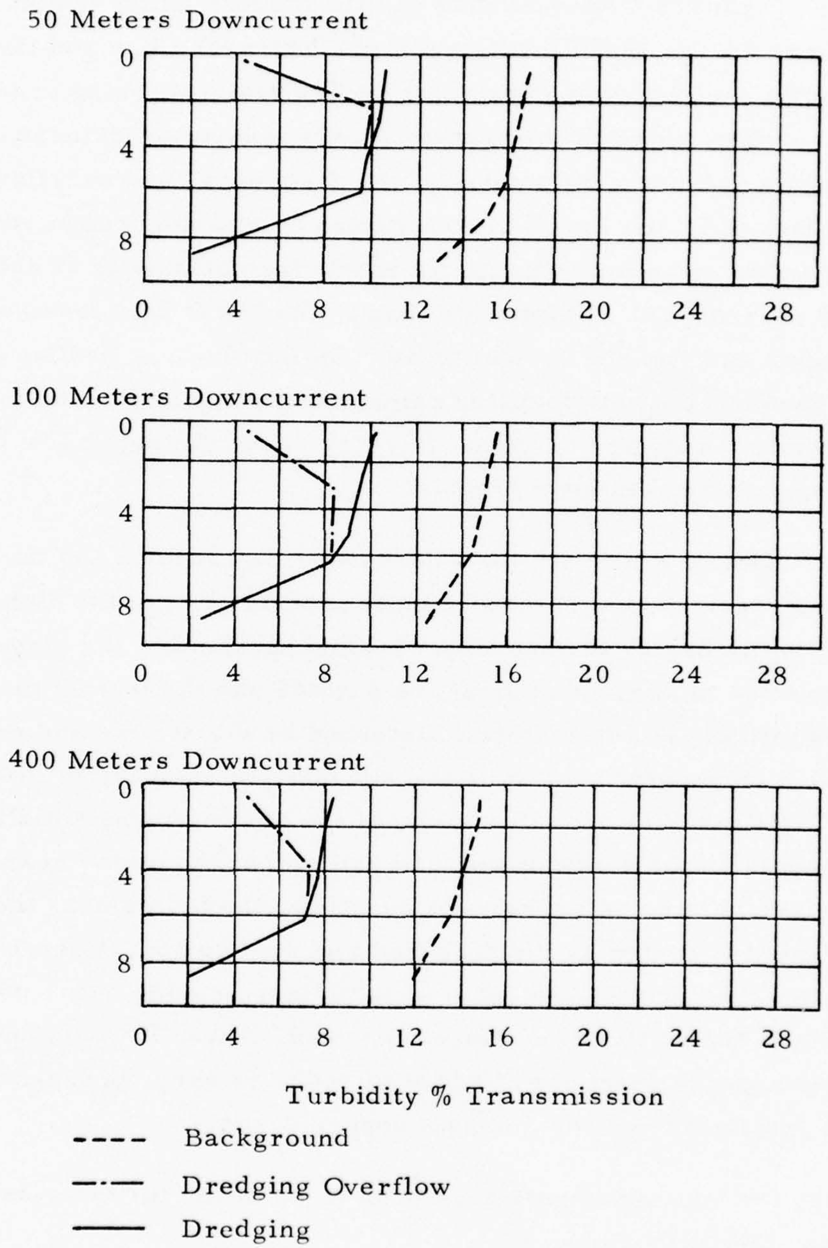


Figure 3-9. Turbidity Generated by Hopper Dredge in San Francisco Bay

near the bottom is more turbid than the overflow plume.

JBF [4] made measurements of the turbidity plume behind the hopper dredge GOETHALS in lower Chesapeake Bay and the results are shown in Figure 3-10. The material being dredged was a silty sand and the transmissivity measured included components due to the twin screws, the dragheads, and overflow. On Day One, at a range of 550 meters behind the dredge, the entire water column was turbid with a transmissivity of about 8-9 percent. At 1100 meters range, the lower half of the water column had cleared substantially. On Day Two, a similar effect is seen but the bottom water remained more turbid than the center of the water column. In every case, the upper few meters is seen to remain highly turbid.

The difference between the Chesapeake Bay results and the San Francisco Bay results is most probably due to the high clay content in San Francisco Bay. In Chesapeake Bay the clay fraction remained in suspension near the surface and the bottom turbidity was initially due to material disturbed by the screws and draghead, plus a component caused by the silt and sand settling to the bottom. In San Francisco Bay, the shape of the curve is substantially retained as the range opens, indicating that the plume near the surface is created by the weir overflow, the plume near the bottom is created by the draghead and screws and, since the material is primarily clay size particles, settling out is not evident during the time frame observed. Thus, it would appear as though the nearfield effect of turbidity is very dependent upon the material type when using a hopper dredge.

Very few measurements have been made of the turbidity associated with other type dredges.

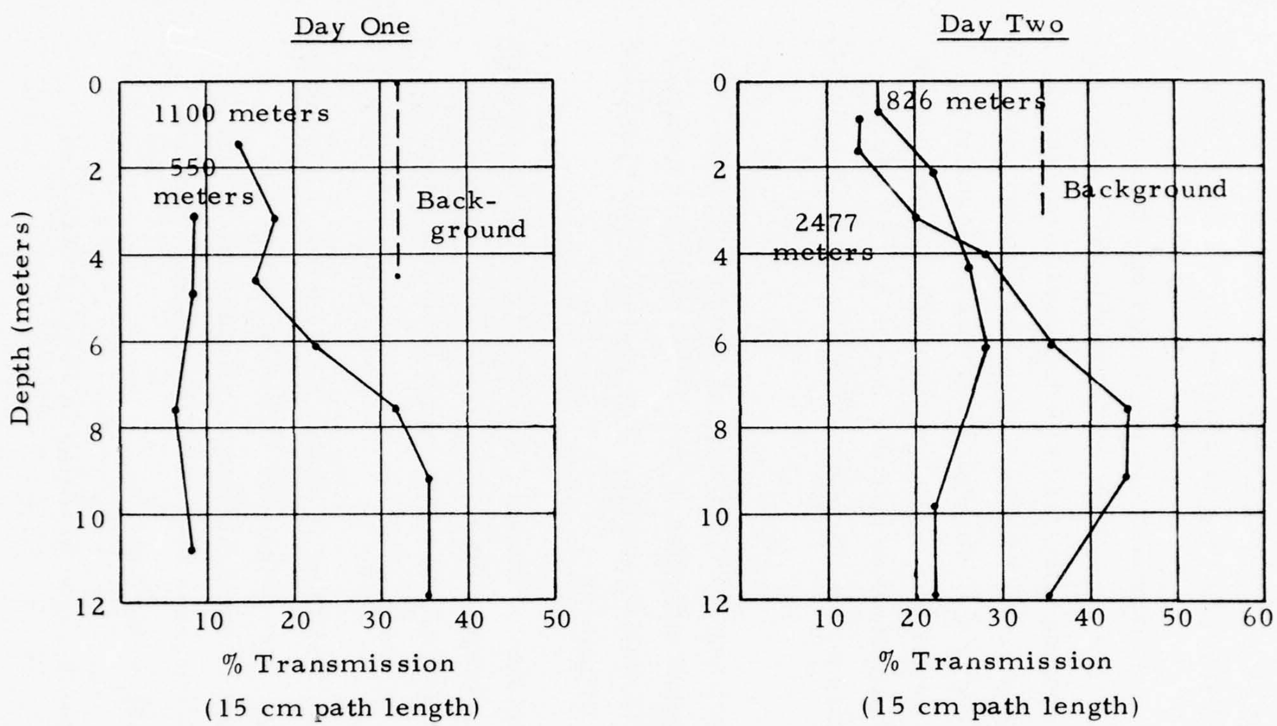


Figure 3-10. GOETHALS Hopper Dredge Turbidity Plume in Chesapeake Bay

Wakeman, et al [3] have made measurements of the turbidity downstream of a clamshell and a cutterhead dredge with the results shown in Figure 3-11. While it is obvious that there is turbidity generated by each type, comparisons cannot be made since the dredges were operating in different areas of the Bay.

Gordon [5] made measurements in the vicinity of a clamshell dredge operating in New Haven Harbor (Connecticut). A 14 cubic yard bucket was dredging silty sand of a high water content of 42 percent; the solids are 35 percent sand, 40 percent silt, and 25 percent clay. Using a 10 cm path length transmissometer, background levels between 70 and 90 percent transmission were measured while transmittance near the dredge was as low as 5 percent. Turbidity was uniform with depth in most of the study area.

The turbidity pattern around the dredge is shown in Figure 3-12 as curves of constant concentration of suspended solids, calculated from the transmissometer records. The contours are identified by weight fraction in units of $10^{-4}$.

Based upon assumptions of the average quantity being dredged by the bucket, and the suspended solids content derived from the transmissometer records, Gordon estimates that the loss rate is 2.5 percent of the solids dug and transferred to the receiving scow.

In a report on gravel dredging in Alabama, Radcliff [6] found that:

> "Gravel dredging produces high turbidity only at the immediate point of spoil discharge. Generally, turbidity was not increased more than 10 JTU above background within 1000 feet of dredging."

A report by Morneault [7] on dredging in Florida indicated:

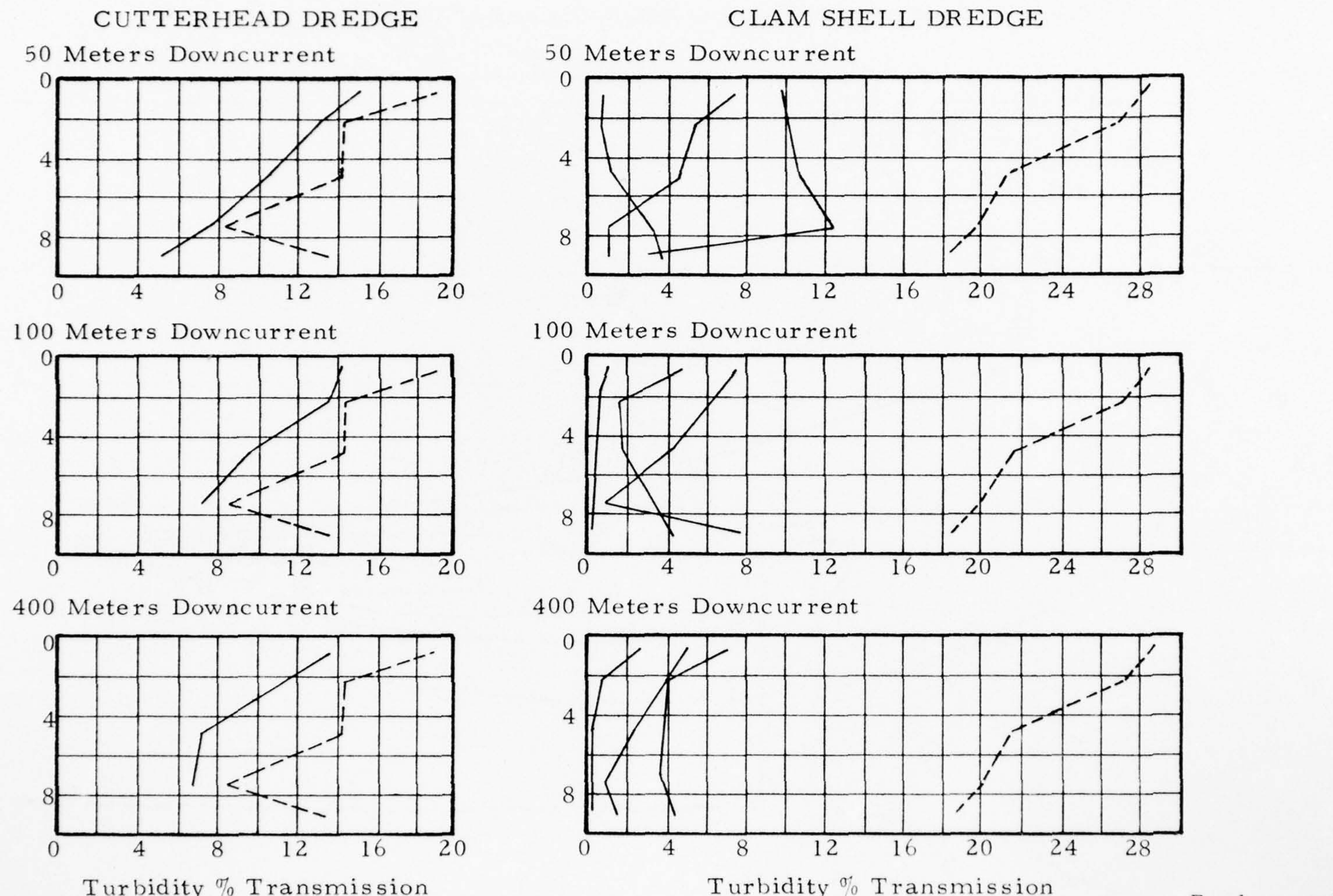


Figure 3-11. Turbidity Generated by Dredges in San Francisco Bay

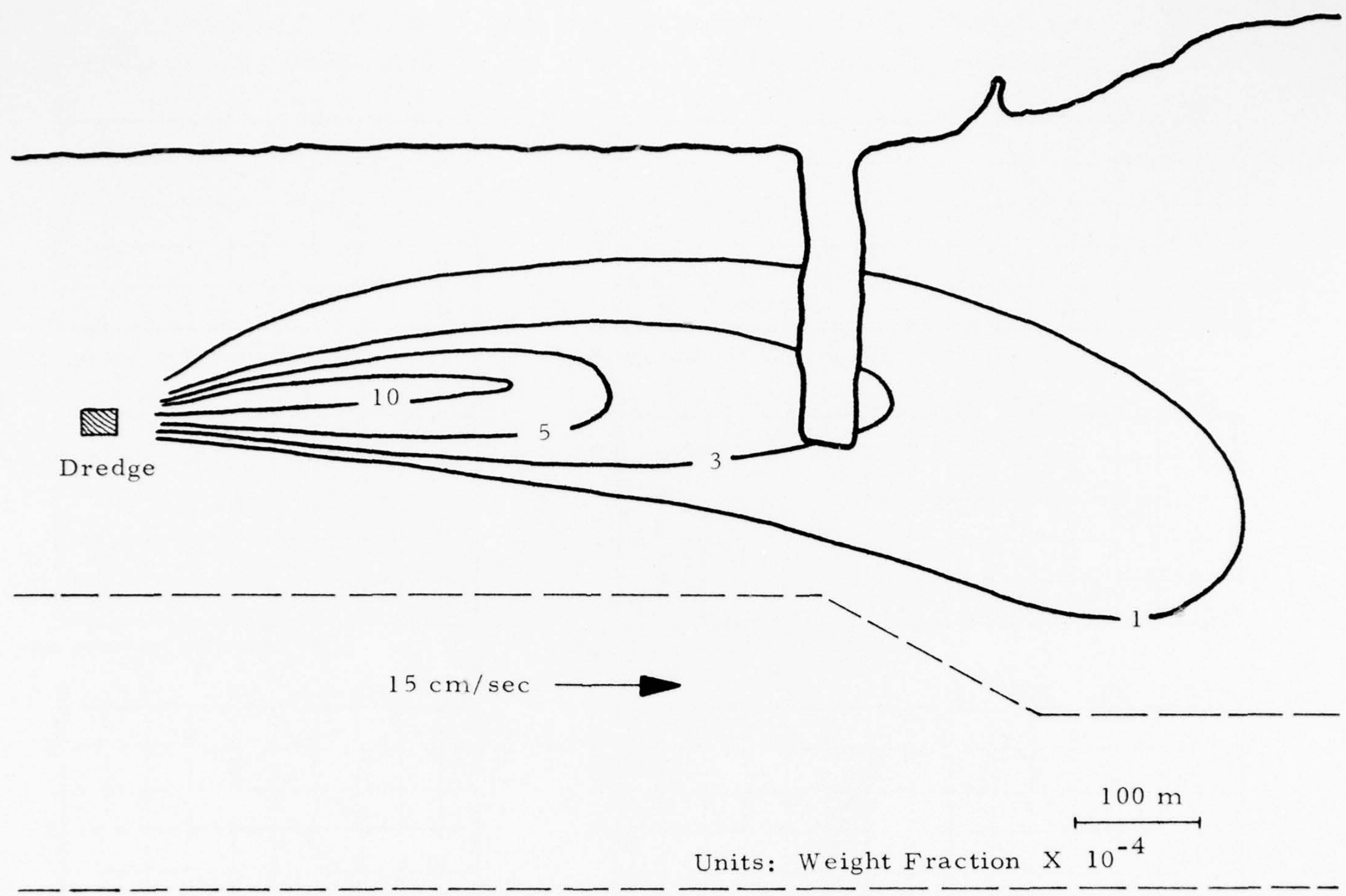


Figure 3-12. Turbidity in New Haven Harbor from a Clamshell Dredge

"Hydraulic dredging equipment utilizes a cutting head and suction line. Material removed by the cutter head is immediately drawn into the suction line limiting silt and turbidity around the cutting head. Thus, the silt plumes around the dredge itself, which are usually associated with dredging, were in this project nonexistent."

Except for these few references, most of the reported work has involved measurements associated with disposal, rather than dredging.

Oxygen Depletion

Oxygen depletion is the other near-field effect of interest to this study. The introduction into the water column of bottom material and pore water from anaerobic in-place sediments usually causes a depression in the oxygen levels in the water column. As in the case of turbidity, oxygen depletion in a dredge plume is highly dependent upon the nature of the sediments and the resident chemical constituents. Thus, it is difficult to generalize about the effects of a given dredging operation.

Wakeman, et al. [3] measured the oxygen depletion in San Francisco Bay waters for three types of dredges. This data is summarized in Figure 3-13. Since the clamshell dredge was operating in Oakland Harbor and the other two dredges in Mare Island Straits, a direct comparison among the three dredges cannot be made. However, there is a similarity between the hopper dredge and cutterhead dredge oxygen depletion and the effect is seen to be quite small for this particular location. Considerable variation is evident in the effects from the clamshell and in some cases the dissolved oxygen levels appear to be actually increased by the dredging operation.

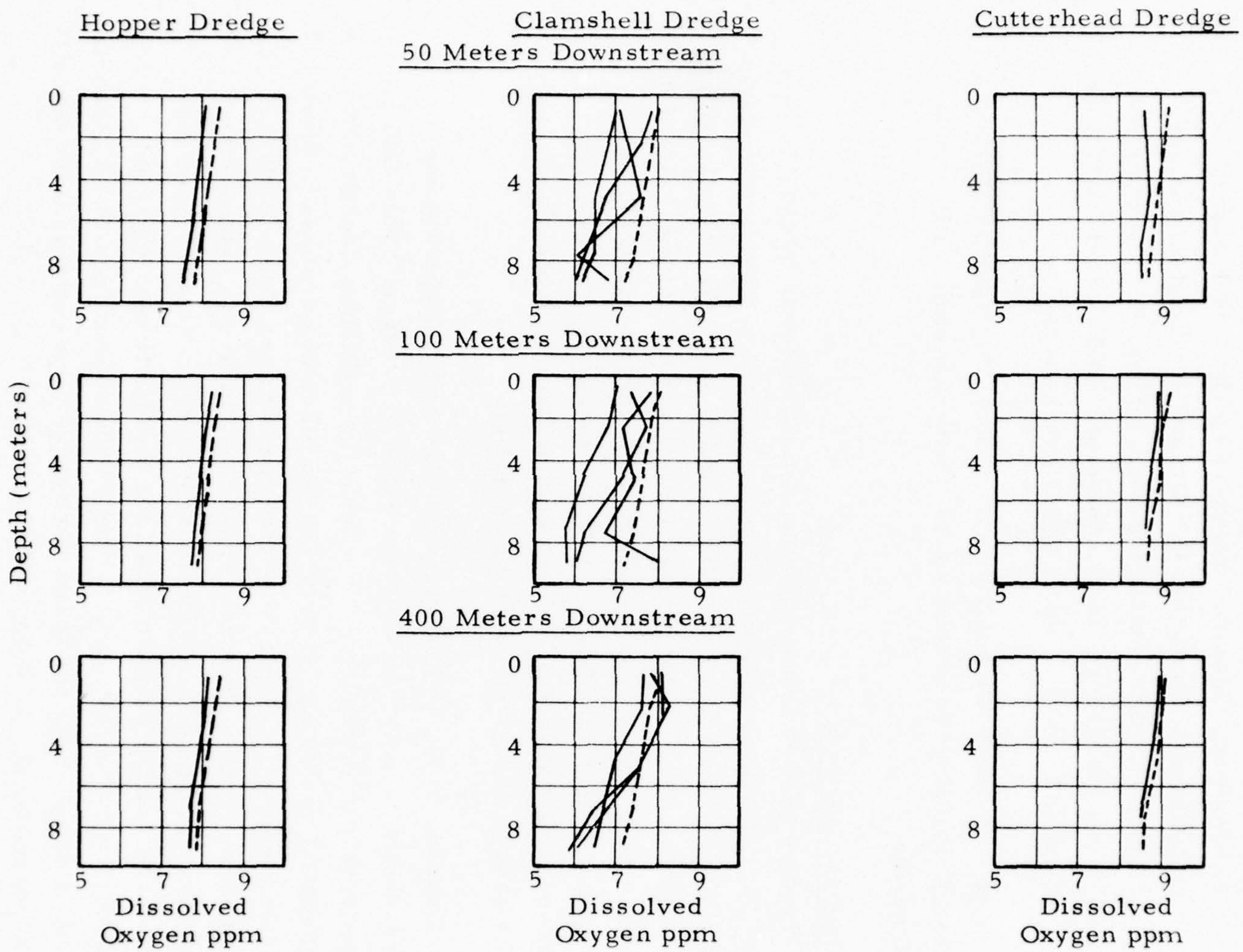


Figure 3-13. Dissolved Oxygen Levels Downstream of Dredges in San Francisco Bay

- - - - - - Background
———— In Plume

JBF made measurements from the clamshell BOSTON, in the approaches to Alameda Naval Air Station, as part of this study. Figure 3-14 shows the effect of the dredging on oxygen levels at two distances downstream of the dredge. The "oxygenation" caused by the dredging is more apparent here than in Wakeman's data. The bucket had a volume of 18 cubic yards and, as it was raised, material was observed to be washing out of the top and draining out of the bottom. The raising of the bucket generated an upwelling in the plume area and this upwelling caused new, highly oxygenated water to be drawn into the plume and exposed the plume surface to the atmosphere. These actions appear, in some cases, to oxygenate the plume volume to such an extent that a depletion does not take place.

Dissolved oxygen measurements were also made in the standing water on top of dredged material in barges filled by the BOSTON and the hoppers of the HARDING. In Alameda, the oxygen levels in the barge were 1.1 ppm at the surface and 0.6 ppm at a depth of three inches. In the hoppers of the HARDING, operating in Mare Island Straits, the dissolved oxygen in the water at the surface of the hoppers was 1.6 ppm.and 1.3 ppm. at a depth of six inches.

After a barge was filled, the procedure used was to clean off the deck prior to transiting to the dump site. Clean water was dumped on the deck using the clamshell until the sediment was washed overboard. This action caused a turbidity plume to be generated. In addition, during the final filling stage the barge often heeled over slightly causing turbid, low oxygen water to spill into the Bay. Measurements were made directly in the plume caused by these activities. The overflow water had a DO of 0.8 ppm; the plume, at a depth of 3 feet, had a DO of 6.6 ppm; and the background _in situ_ water had a DO of 8.0 ppm.

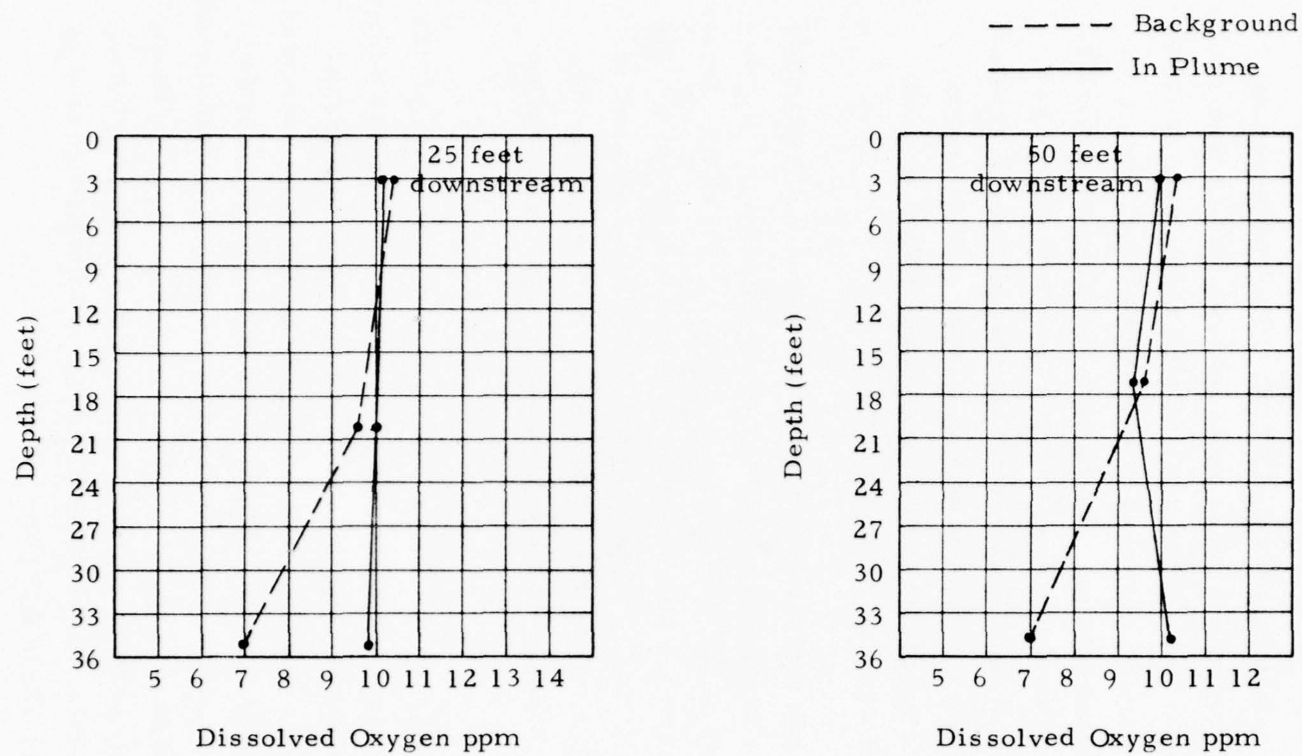


Figure 3-14. Dissolved Oxygen Levels in Alameda Using a Clamshell Dredge

## 4. Effect on Sediment Physical Characteristics

The dredging operation takes in-place material and, in some manner, removes it from the bay and places it in a hopper, a barge, or a landfill area. Depending on the dredging mode used, the effect on sediment characteristics may be large or small.

JBF conducted field studies in San Francisco Bay to establish the effect on the characteristics of sediments due to the dredging operation, using scuba divers, a clamshell dredge, and a hopper dredge. For continuity, the results will all be reported in this section even though some of the measurements were made during the transport phase.

The characteristics to be studied were selected based upon *a priori* considerations and included: shear strength, penetration, density, water content, grain size, and oxygen demand. While the latter is not a physical characteristic, it was included because of its importance during the disposal phase in open water dumping.

Measurements were made *in situ*, during the filling operation, and after the material was in the transport vessel, as described in an earlier section of this report.

### Oxygen Demand

Oxygen demand was established by measurements of IOD and COD. IOD was measured by a technique based on a *Standard Methods* [1] procedure in which the level of dissolved oxygen in a closed bottle is observed as a function of time. The addition of a known amount of dredged material causes the DO level to drop because of oxidation of reduced minerals, predominantly iron and manganese. The time during which DO levels were observed was a maximum of 60 minutes. In a number of cases duplicate analyses were made to establish the precision of the technique. Figure 3-15 shows typical data from which the IOD was determined.

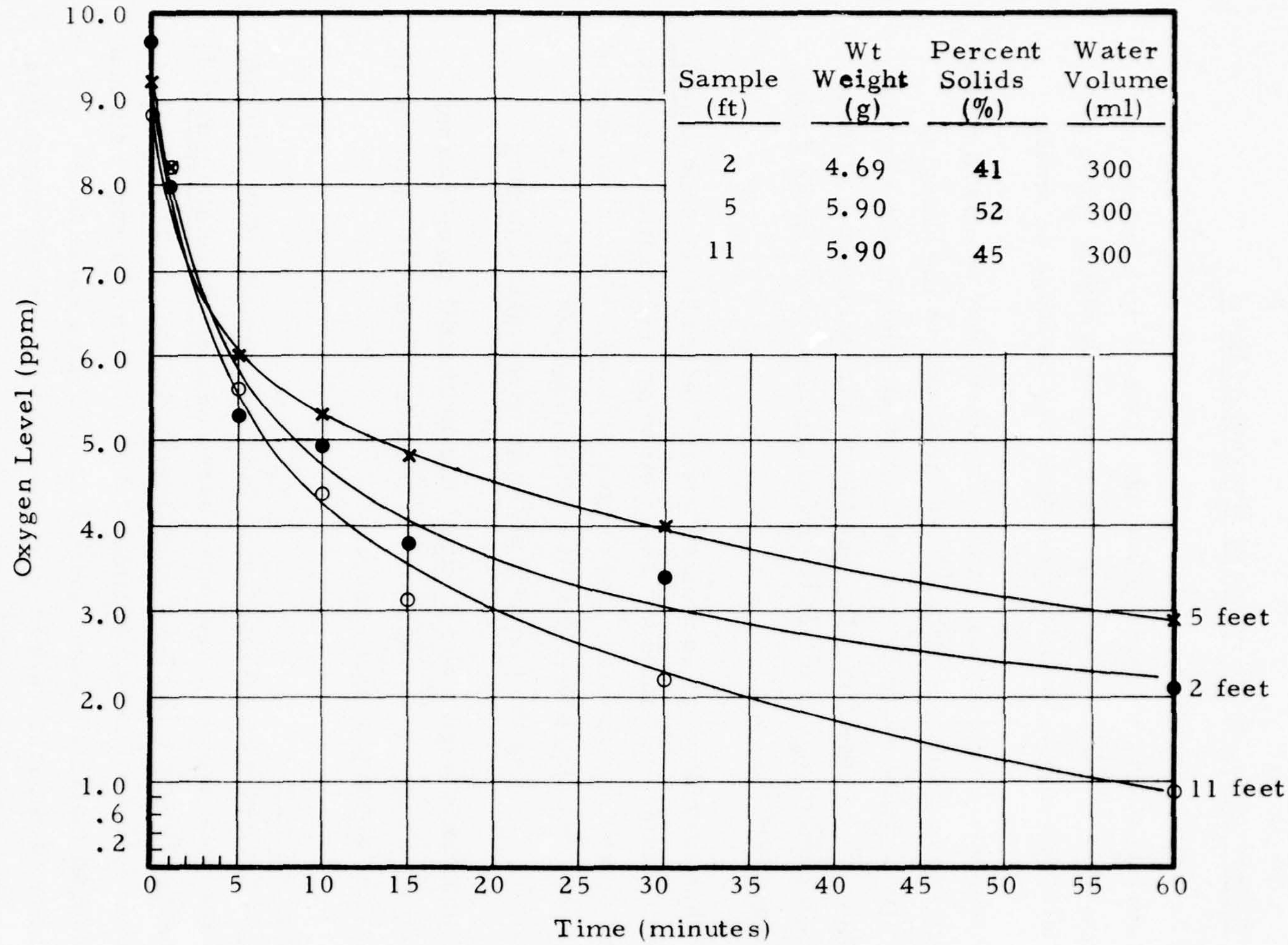


Figure 3-15. IOD Measurement in Barge Samples from 3 Depths

Oxygen demand was measured on material obtained in situ, from the clamshell bucket, the barge, and the hopper dredge. The primary intent was to discover what discernible effects dredging and transport might have on oxygen demand of the dredged material.

Variability among samples taken in both dredging operations was relatively high although the repeatability of the IOD procedure was excellent reflecting the non-homogeneity of the material recovered. Since the variability was large, minor effects on oxygen demand may have been masked; however, the data showed no major changes in oxygen demand due to the dredging operations.

(a) Clamshell Dredge

The IOD and COD measurements taken at the clamshell dredging sites are summarized in Tables 3-6 and 3-7. As a guide to interpreting this information it should be noted that the standard deviations for replicate analyses of the same sample were about 36 g /kg and less than 2 mg/kg for IOD and COD respectively. Variations among samples taken at the time are shown in Tables 3-6 and 3-7 and the standard deviations average about 150 mg/kg for IOD and 4 g /kg for COD. Thus, variability of the samples from the clamshell and the barge are considerably more than the variations due to sampling method and analysis. This reflects the non-homogeneity of the material.

It should also be noted that the data presented in Tables 3-6 and 3-7 are from different dredging sites and therefore may reflect site differences. However, the site-to-site variability in data for Alamada does not appear to differ significantly from the "within-site" variability. For this reason data was combined for all three Alameda sites to provide larger samples for recognizing trends.

The COD information indicates a slight, but perhaps significant, trend towards lowering of oxygen demand by the dredging operation. The COD level of material in the clamshell or in the barge shows

TABLE 3-6 - COD Measured in Bucket Dredge, g/kg

| | in situ | In the bucket | In the barge |
|---|---|---|---|
| Mean | 55 | 49 | 48 |
| Standard Deviation | 1.6 | 5.7 | 4.6 |
| Number of Samples | 4 | 4 | 6 |

TABLE 3-7 - IOD Measured in Bucket Dredge mg/kg

| | in situ | In the bucket | In the barge |
|---|---|---|---|
| Mean | 711 | 1028 | 940 |
| Standard Deviation | 98 | 161 | 179 |
| Number of Samples | 3 | 3 | 7 |

greater variability than that in situ, perhaps due to some satisfaction of the oxygen demand taking place as the material became well dispersed during the dredging operation. However, the mean COD levels show only slight reduction from in situ means, indicating that very little of the COD is satisfied by the clamshell dredging operation.

The IOD information shows negligible change from the clamshell samples to the barge samples and the in situ IOD levels are, in fact, lower than those in the bucket or barge. The probable explanation is that the in situ samples were of surface mud at the harbor bottom and did not represent typical material at the grade level of the dredging operation. The material at the grade level may have had a higher oxygen demand to account for the levels observed in the clamshell and in the barge. We feel, however, that no significant satisfaction of IOD takes place as a result of the clamshell dredging operation.

(b) Hopper Dredge

The IOD and COD measurements taken at the hopper dredging site are summarized in Tables 3-8 and 3-9. These data include in situ measurements as well as measurements of oxygen demand in the mud flow and in the hopper itself. Because of the method of operation of the dredge the mud flow varied between a "heavy density" flow of low water content to a "light density" flow of high water content. Once in the hopper, the heavy material sank to the bottom leaving a surface layer of turbid water. Since the oxygen demand appeared to differ significantly according to the flow density and the depth in the hopper, the data has been separated on these parameters.

The COD data shows a negligible difference between the in situ samples and the hopper samples. The differences between the

TABLE 3-8 - COD Measurement in Hopper Dredge g/kg

| | MUD FLOW | | | IN HOPPER | |
|---|---|---|---|---|---|
| | in situ | high density | low density | (20') deep | (20') shallow |
| mean | 50.5 | 45 | 49 | 49.9 | 51.2 |
| standard dev. | 1.6 | | | | |
| no. in sample | 6 | 1 | 1 | 1 | 2 |

TABLE 3-9 - IOD Measured in Hopper Dredge mg/g

| | MUD FLOW | | | IN HOPPER | |
|---|---|---|---|---|---|
| | in situ | high density | low density | (>20') deep | (<20') shallow |
| mean | 875 | 921 | 513 | 867 | 690 |
| standard dev. | 197 | | | | 139 |
| no. in sample | 4 | 2 | 2 | 2 | 3 |

high density flow and low density flow and the differences with hopper depth also are not significant. The depressed values found in the mud flow (as compared to *in situ*) might reflect dilution of the material by water drawn by the pump. In general, these data show no evidence of satisfaction of the COD by the hopper dredging operation.

The IOD data shows little change between the high density flow and the "deep" hopper samples, or between the low density flow and the "shallow" samples, indicating that the heavy material taken from the harbor bottom sinks into the hopper with no significant reduction in oxygen demand. Again the *in situ* measurements of IOD are low, probably reflecting the fact that samples obtained by the divers were not taken sufficiently deep in the mud to represent the bulk of the dredged material. It appears that no significant reduction of IOD results from the hopper dredging operation.

It can be demonstrated that very little of the IOD initially present in the sediment will be satisfied during the dredging process with a hopper dredge. This may be seen by noting that typical IOD values found during this study were about 1000 mg/kg. Assume that the sediment were diluted during the dredging operation such that the concentration of IOD in the dredged material slurry was 500 mg/l. The dredged material slurry would initially have a DO of 5 mg/l if the overlying water had a DO of 10 mg/l (the sediment DO will be zero). Thus, even if the dissolved oxygen were completely reacted with the IOD, the reduction in IOD would be only 5 mg/l or 1 percent of the initial IOD.

As the dredged material slurry enters the dredge, it travels down chutes and splashes into the hoppers. During this period oxygen from the air will react with IOD. However, once in the hopper the

sediment will rapidly settle below the water surface and be much less available to oxygenation reactions. The extent of oxygenation during the time from leaving the pipe until deposited in the hopper can be estimated with an IOD model developed by JBF.
The model assumes first order reaction kinetics and reoxygenation of the slurry water from the atmosphere. Allowing reasonable assumptions for the physical constants and 1.0 minute of contact with the air while traveling in the chute and splashing into the hopper, the model predicts that about 9 mg/l of IOD will be satisfied. Thus, from the time of dredging to deposit in the hopper, only 14 mg/l of IOD from an original 500 mg/l, or about 3 percent, will be satisfied. While some uncertainty may exist concerning the exact numbers, it is clear that the hopper dredging operation will not substantially affect the sediment IOD.

Since the exposure to oxygenated water is less for a clamshell, the effect would be even smaller. In the case of a hydraulic pipeline dredge, the retention times in the pipeline are longer and more oxygen demand can be satisfied. However, any significant satisfaction of this IOD will require the addition of bulk oxygen to the pipeline.

Based upon the measurements made and the above analysis, there is no reason to believe that a routine dredging operation can significantly affect the oxygen demand of the dredged material regardless of the type dredge used, unless the sediment oxygen demand is extremely low, a case of little interest to this discussion.

Moisture Content and Dry Density

Moisture content was measured in situ, in the clamshell, in the barge filled with a clamshell, and in the hopper dredge.

The data is presented as percent moisture, percent solids, and dry density. Values were calculated as follows:

$$\text{Percent Moisture (PCM)} = \frac{W_w}{W_s} \times 100$$

$$\text{Percent Solids (PCS)} = \frac{W_s}{W_s + W_w} \times 100$$

$$\text{Dry Density} = \frac{W_s}{V}$$

where: $W_w$ = weight of the water

$W_s$ = dry weight of the solids

$V$ = total sample volume

Percent moisture and percent solids are related by:

$$PCS = 100\left[\frac{1}{1 + \frac{PCM}{100}}\right]$$

In this section both PCM and PCS are plotted for the readers convenience. Dry density data is presented in the data tables in Appendix A.

Samples were taken in a number of pockets in the barge, and at several depths, in both the barge and the hopper operation. In the hopper operation, samples were also taken at various times during the run from the dredging site to the disposal site. The primary intent was to discover what effect the method of dredging and transit to the dump site had on moisture content of dredged material.

In both the hopper and the barge pockets, the moisture content was

at a maximum at the surface and diminished with depth. The data exhibited greater variability when the water content was high. For barges filled with a clamshell, the distribution of moisture content in the pockets exhibited a dependency on the order in which the pockets were filled.

(a) In Situ Measurements

Divers collected samples in Alameda and in Mare Island Straits in the areas being dredged and at the Mare Island site, just north of the dredged area. In each case the dive vessel was anchored and the divers went down the anchor line.

In Mare Island Straits the divers moved freely down the line until almost at the "bottom," at which time the diver could feel the density of the fluid increasing. Near the water/sediment interface there appeared to be a substantial gradient in density, or viscosity, and the divers could not reach the depth of the anchor without their masks filling with sediment and their regulators free flowing. However, the anchor could be felt with the divers feet when they sank vertically into the sediment.

A water/sediment interface could be located by the diver slowly moving his hand vertically with the palm horizontal. At a depth of 12 to 18 inches below this interface, his regulator would free flow. Measurements were made on samples obtained by the divers at depths above and below the interface. It should be realized, however, that the depths at which these samples were obtained could vary by as much as 6 to 8 inches due to the hostile working conditions.

Figures 3-16 and 3-17 show plots of the gradient in Mare Island Straits and Alameda for data obtained on diver samples. At Alameda, a definite water/sediment interface existed and the divers could only obtain samples to a depth 18 inches below it, whereas in Mare Island Straits they could penetrate 36 inches.

This suggests that the gradient below the interface is much steeper at Alameda than Mare Island Straits.

Two samples were obtained several hundred feet north of the dredged area in Mare Island Straits. The average percent moisture for these two was 197 percent. Since these samples were obtained below the depth at which the divers regulator free flowed, this suggests that the gradient in the undredged area may not be as steep as the gradient in the area being dredged. The divers described the undredged area as feeling "silty," or "spongy," and said that the sampler felt as if it would come back up when pushed into the sediment. They described the dredged area as being "sticky," or "claylike," and being firmer. They could stand up in the dredged area but could not stand in the undredged area.

A number of samples using a variety of samplers, were obtained in Alameda, as shown in Table 3-10. Except for the 2 inch diameter PVC tube, the results obtained are quite similar. The 2 inch diameter tube was approximately 30 inches long and was pushed into the sediment as deep as the diver could reach. Thus, the top reading undoubtedly describes sediment close to the interface, as shown in Figure 3-17, and the bottom reading is probably the deepest penetration obtained by the divers. Since it is higher in solids than the 18 inch sample, this suggests that the percent solids gradient is still increasing at depths greater than 18 inches below the interface.

(b) <u>Clamshell Dredge</u>

The observed relationship between moisture content and depth within a barge pocket is illustrated in Figure 3-18 which shows depth vs percent moisture and percent solids for a series of samples taken in a single barge pocket for a single dredging run.

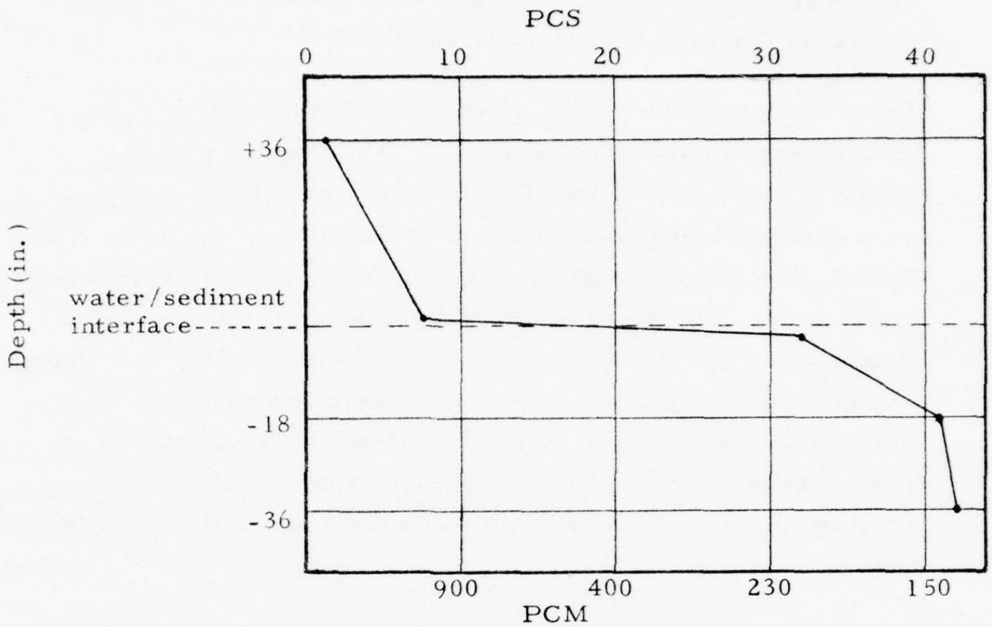


Figure 3-16. In Situ Measurements of Moisture Content Gradient in Mare Island Straits

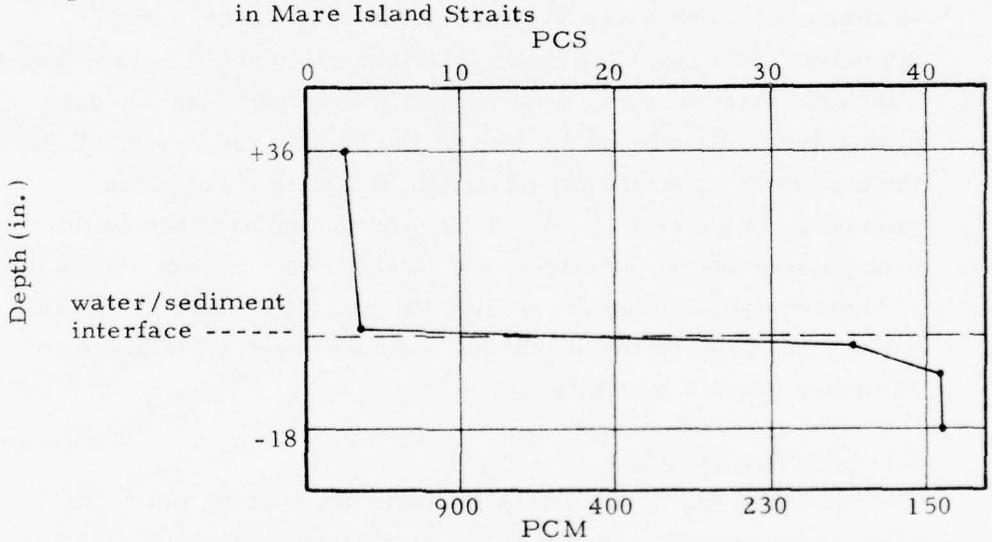


Figure 3-17. In Situ Measurements of Moisture Content Gradient in Alameda

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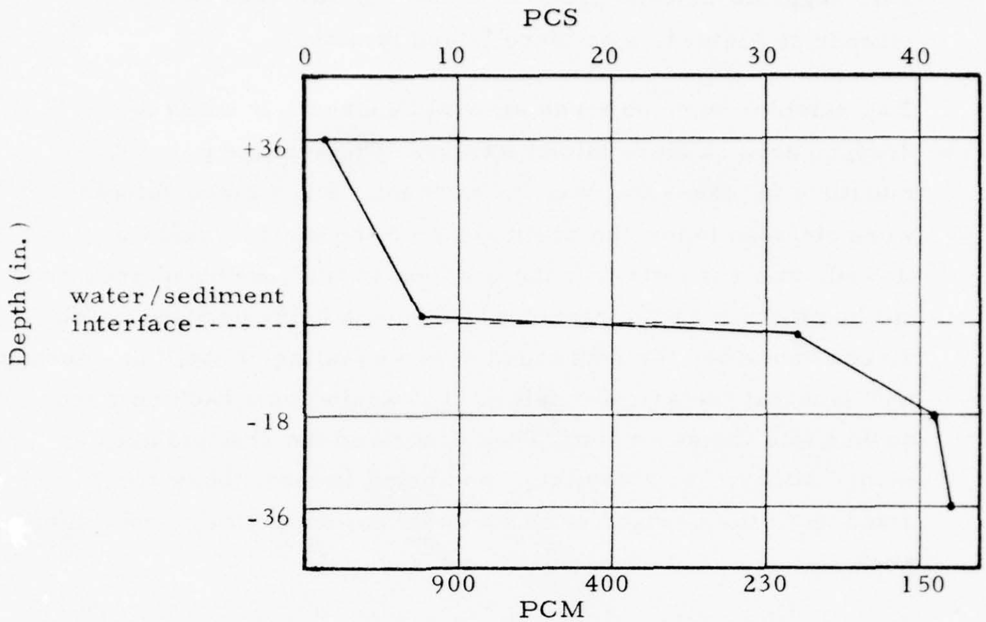


Figure 3-16. In Situ Measurements of Moisture Content Gradient in Mare Island Straits

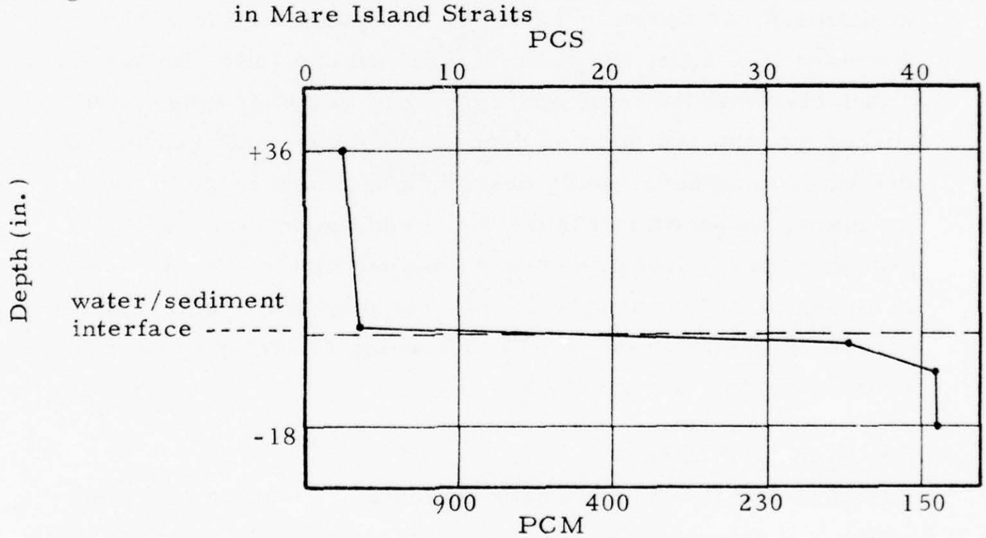


Figure 3-17. In Situ Measurements of Moisture Content Gradient in Alameda

TABLE 3-10 - Data From Diver Samples in Alameda

| | | % Moisture | % Solids |
|---|---|---|---|
| 6 in. diameter PVC tube | | 148 | 40.3 |
| 2 in. diameter brass tube | | 157, 153 | 38.9, 39,5 |
| 3 in diameter PVC tube | | 142 | 39.2 |
| 2 in diameter PVC tube | top | 173 | 36.6 |
| | bottom | 124 | 44.6 |
| | | 149.5 | 40.1 |

The high water content on the surface and the slight trend towards decreased moisture content at greater depth is evident from these data although the spread of the data is quite large. In interpreting this data it should be noted that the standard deviation due to replication of individual sampls produced a variation of about 15 percent due to analytical procedures. This is nearly as large as the observed standard deviation in the data samples. Thus, much of the data variation about the line of moisture content vs depth is reflecting sampling error, which includes variation due to the sampling method and the analytical procedure used. We would expect the latter to be very small.

Figure 3-19 shows the moisture content in a number of barge pockets for samples at two depths. In this case the barge was initially loaded in the center pockets (4 and 5) and hence the increased moisture content, particularly at the surface, in the end pockets (1 and 6) is expected, due to excess water running into the end pockets. The data shown in Figure 3-19 was taken while the dredge was filling a single barge. The data demonstrates an effect on moisture content of the recovered material, due to the method of loading the barge (i.e., center pockets first).

If the data in Table 3-10, showing the percent moisture from <u>in situ</u>

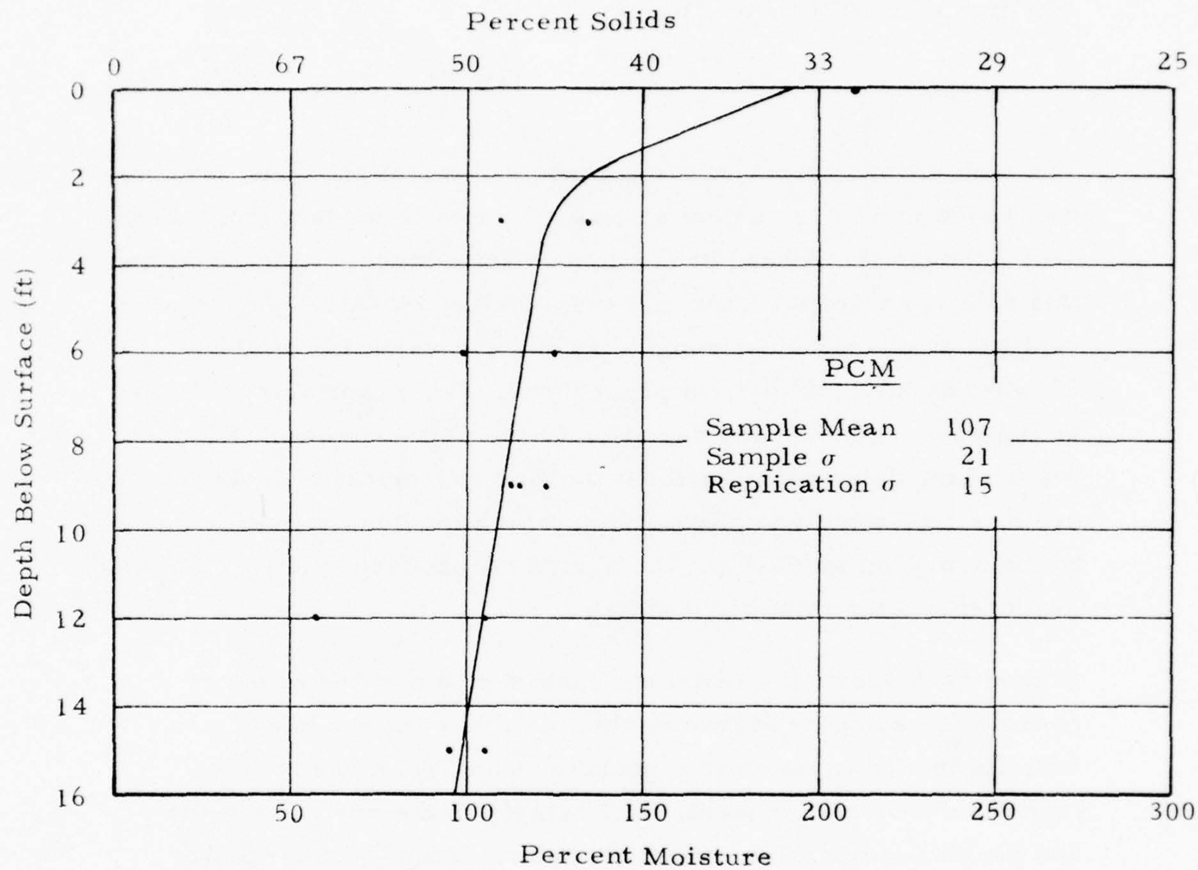


Figure 3-18. Moisture Content as a Function of Depth in a Barge Pocket

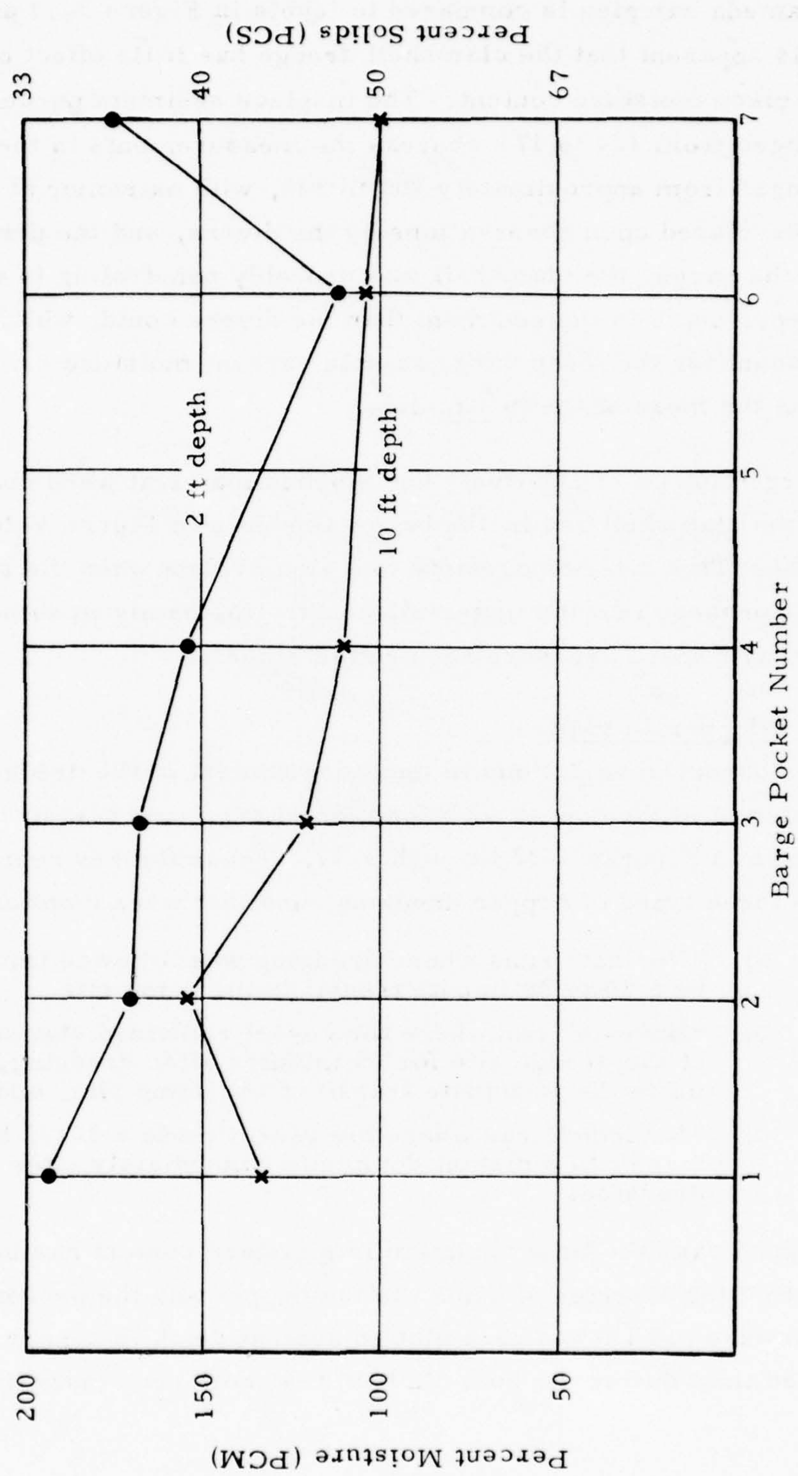


Figure 3-19. Variation in Barge Pocket Moisture Content as a Function of Depth

Alameda samples is compared to levels in Figure 3-18 and 3-19, it is apparent that the clamshell dredge has little effect on the in-place moisture content. The in-place sediment percent moisture ranged from 124 to 173 whereas the measurements in the barge ranged from approximately 100 to 140, with extremes at 70 and 210. Based upon observations by the divers, and the personnel on the barge, the clamshell was probably penetrating to a deeper depth in the sediment than the divers could, which would account for the mean barge sample percent moisture being lower than the mean of the in situ data.

Large clumps of relatively undisturbed material were observed in the clamshell and in the barge as shown in Figure 3-20 and 3-21. This non-homogeniety was also evident when the sampler was pressed into the material, penetrating easily in some places and with more pressure required in others.

(c) Hopper Dredge

The observed variations in moisture content of the dredged material in the hopper as a function of depth and transit time are shown in Figures 3-22 through 3-27. These figures represent the three types of hopper dredging runs that were monitored:

a) "Normal" runs where dredging was followed immediately by a 10 to 20 minute transit to the dump site

b) "Hove-to" run where the vessel remained stationary at the dredge site for 15 minutes after dredging, then made the 15 minute transit to the dump site, and

c) "Extended" run where the vessel made a 2-1/2 hour transit to a distant dump site immediately after dredging.

In each case the time variation in moisture content has been plotted for a series of depths in the hopper and the percent moisture profile has been plotted against depth in hopper for a fixed time during the run. In both the short runs (normal and

Figure 3-20. Dredged Material in the Clamshell

Figure 3-21. Dredged Material in the Barge

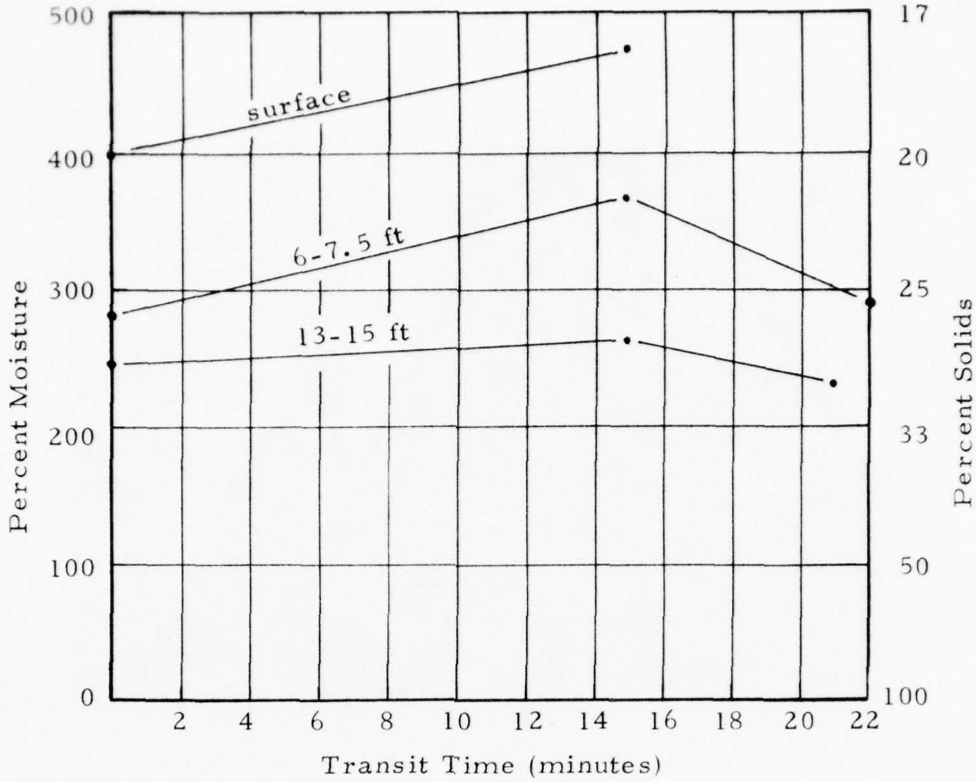


Figure 3-22. Hopper Moisture Content for Transit to Nearby Dump Area

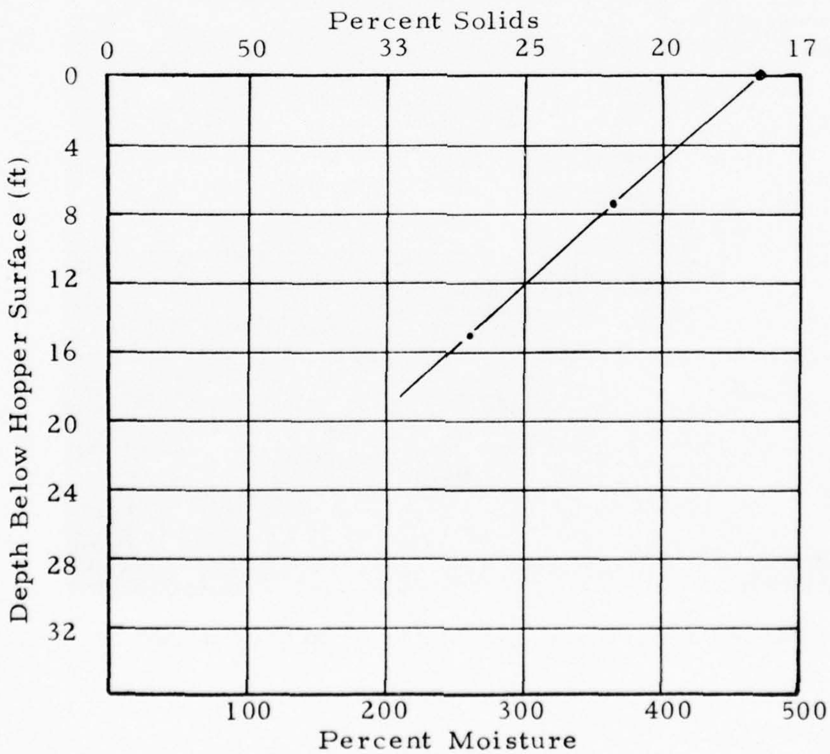


Figure 3-23. Moisture Content Distribution in Hopper After 15 Minutes of Transit

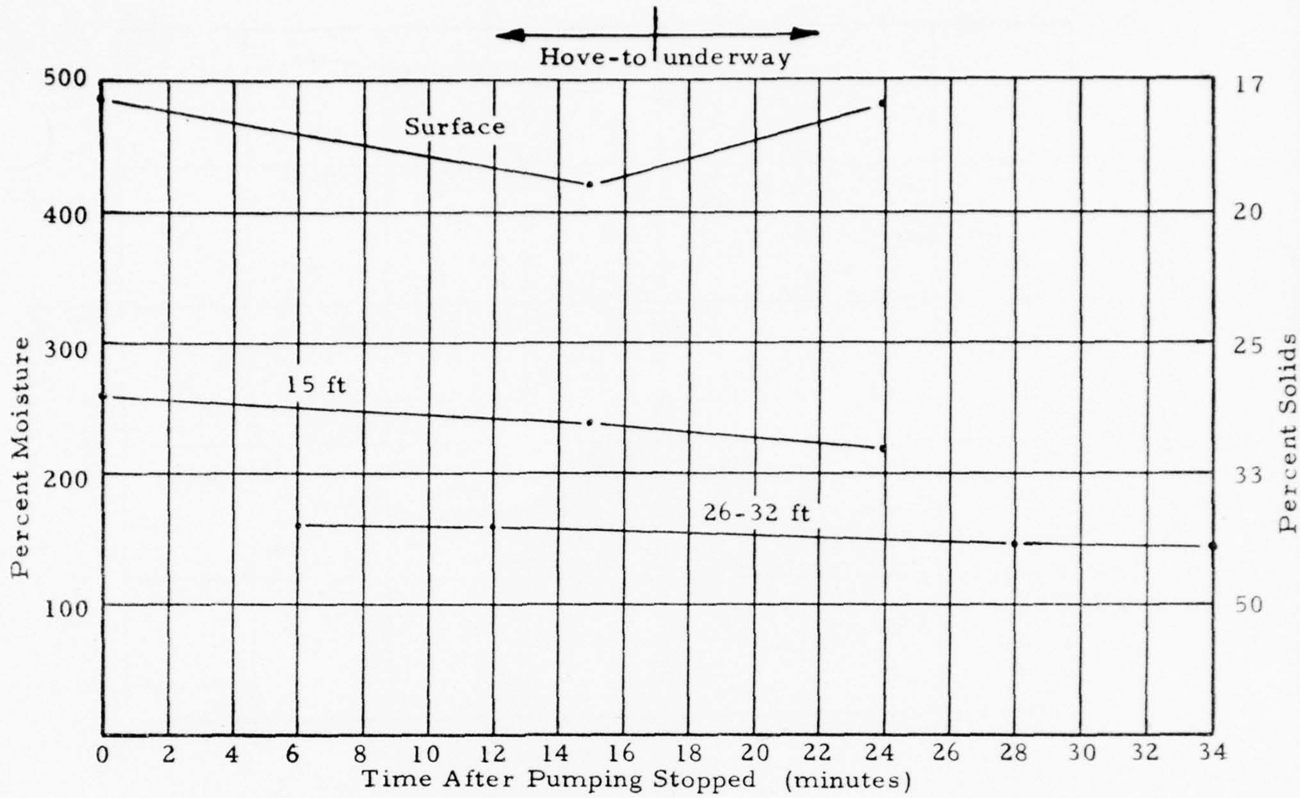


Figure 3-24. Moisture Content of Hopper Under Hove-to and Transiting Conditions

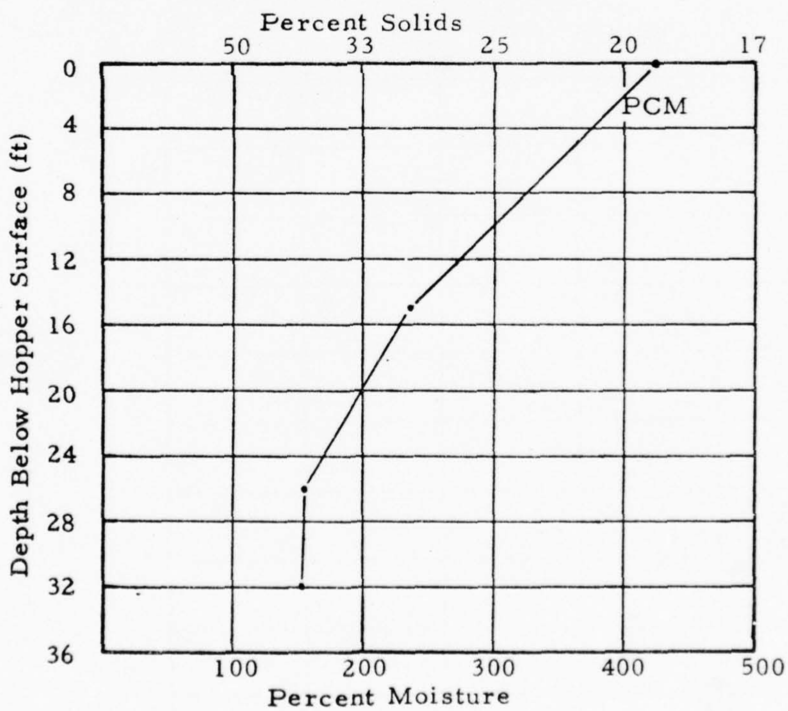


Figure 3-25. Moisture Content in Hopper After Being Hove-to 15 Minutes

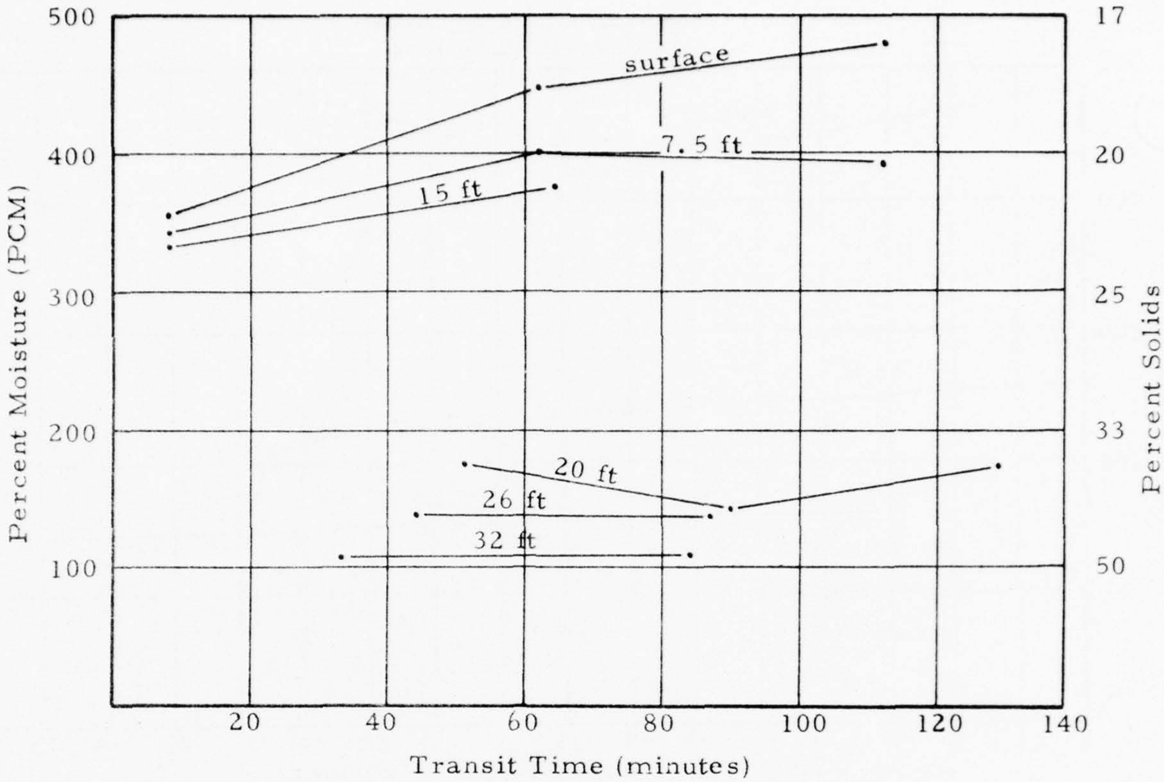


Figure 3-26. Moisture Content in Hopper During Extended Transit to Dump Site

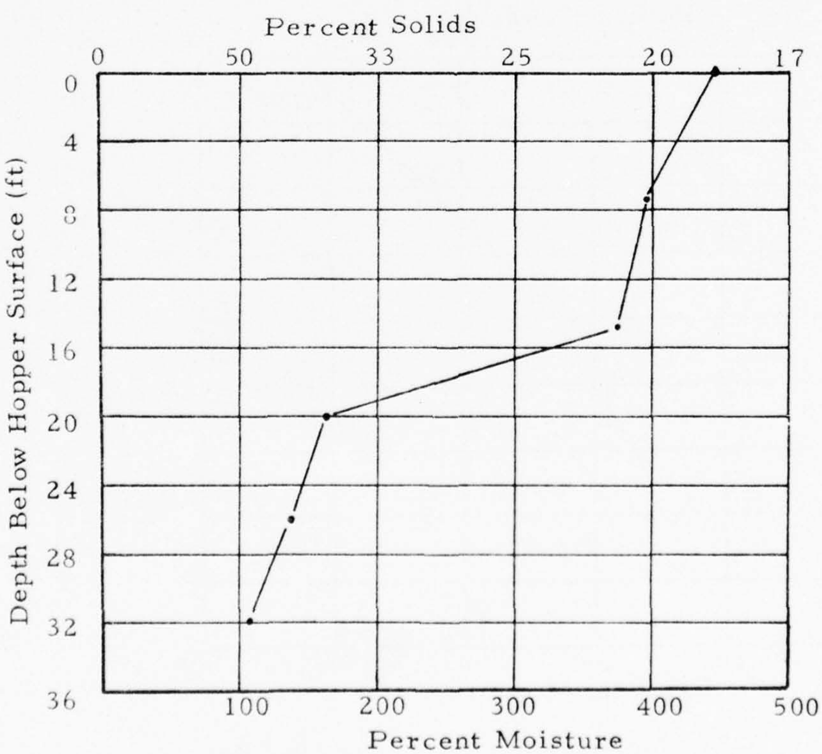


Figure 3-27. Moisture Content Distribution in Hopper After 65 Minutes of Transit

hove-to runs) the decrease in moisture content with increased depth is apparent; however, there is no indication of a stratification into layers in the hopper. In the extended run, on the other hand, a stratification of the material into a high solids layer in the hopper at depths below 15 feet and a low solids layer in the upper 15 feet is apparent after about 60 minutes running time. The lower layer appeared to be about 40 to 50 percent solids while the surface layer is increased in moisture content to about 20 percent solids after 1 hour running time.

If the in-place sediment for Mare Island Straits, from Figure 3-16, is compared to the graphs of moisture content in the hoppers, it is found that the material in the lower layer is of approximately the same moisture content as in situ. For a 15 minute residence time (as in dumping in Carquinez Straits), this is true at depths in the hopper greater than approximately 20 feet (about half the hopper depth). After a 65 minute residence time, the extent of low moisture content material has increased by several feet.

In general, the moisture content in the material in the hopper dredge appears to be higher than that in the clamshell operation although after stratification, the material near the bottom of the hopper approaches the percent solids found in the clamshell operation. However, it must be remembered that the clamshell and hopper operation took place at different sites and, therefore, site-to-site differences may have affected this comparison.

As can be seen from the figures the effect of transit time on moisture content of the dredge material is not large. Only in the extended run does this effect appear to become significant and then not until after about one hour in transit and only affects the upper half of the material in the hopper. In the short runs (normal and hove-to) the effect is not significant for either the

hove-to or the underway time, indicating that the underway vibrations are probably not significant for this parameter. However, the increase in percent solids, (or decrease in percent moisture) with depth in the hopper is significant in all runs.

Near the surface of the hopper the dredge material is highly non-homogeneous and consists of patches of water and clumps of solid material. This non-homogeneity is reflected in the analysis of surface samples where percent solids vary from about 5 percent to over 20 percent. These figures can be compared with an analysis variation of about $\pm$ 2 percent solids. This non-homogeneity is not evident in Figures 3-22 to 3-27 since these data are averages derived from mixing sample material thoroughly before drawing the analysis sample. This mixing has the effect of reducing the variation due to non-homogeneity of the surface dredge materials.

The data samples in the hopper dredging operation were taken by both the grab sampling method and the syringe sampler method shown in Figure 3-5. The grab samples were used for sampling in the upper 15 feet of the hopper and the syringe sampler method was used below 15 feet depth. There was evidence of an effect due to method of sampling with the syringe sampler method giving somewhat greater percent moisture than the grab samples for similar samples. The sampling effect was greatest when moisture content was high (about 100 PCM difference at 300 PCM). This effect was not as large below 15 feet depth, where moisture content was lower, and therefore it did not mask the effect of depth in hopper. Tests on the syringe and controlled volume <u>in situ</u> samples indicated that for sediments of the percent moisture found in the bottom half of the hopper, or the barge, the effect was unimportant. For this reason, tests reported in the upper 15 feet of the hopper are from samples obtained with a

standard 2 inch diameter water sampler and syringe sampler was only used for depths greater than 15 feet.

Figure 3-28 shows the nature of the material in the hopper upper region, at a depth of approximately 7.5 feet.

(d) Shear Strength

Figure 3-29 shows a typical result of the field vane shear tests aboard the Hopper Dredge HARDING. Additional graphs are included in Appendix B. The graph shows the calculated shear strength in psf versus degrees rotation of the shear vane, with the values of shear strength ranging from 0 to 8 psf after a correction for rod friction was made (see Appendix B for a discussion of rod friction values and for data on shear vane calibrations.)

In some cases it was found that the values obtained were smaller than the assumed rod friction and that the net shear strength was therefore zero. A number of the graphs in the Appendix also show residual shear strength values, which are not corrected for rod friction. The residual values were obtained by rotating the field vane a full $360^{o}$ and then recording the resistance past $360^{o}$, when the vane was pushing against remolded materials. The location, depth, and time of each test is also shown on the figures.

Figure 3-30 shows shear strength vs degrees rotation measured aboard a barge filled by the clamshell dredge BOSTON. Table 3-11 presents a summary of the shear strength measurements made on the two dredges. The corrected values of shear strength run as high as 110 pcf on the clamshell. While these values are substantially higher than the values on the HARDING, they are not as consistent. The basic difference in the method of dredging, the pickup of the sediment by clamshell vs the drag head of the hopper dredge, explains this difference in apparent shear strength.

Figure 3-28. Nature of the Material in the Hopper at a Depth of 7.5 feet Below the Surface

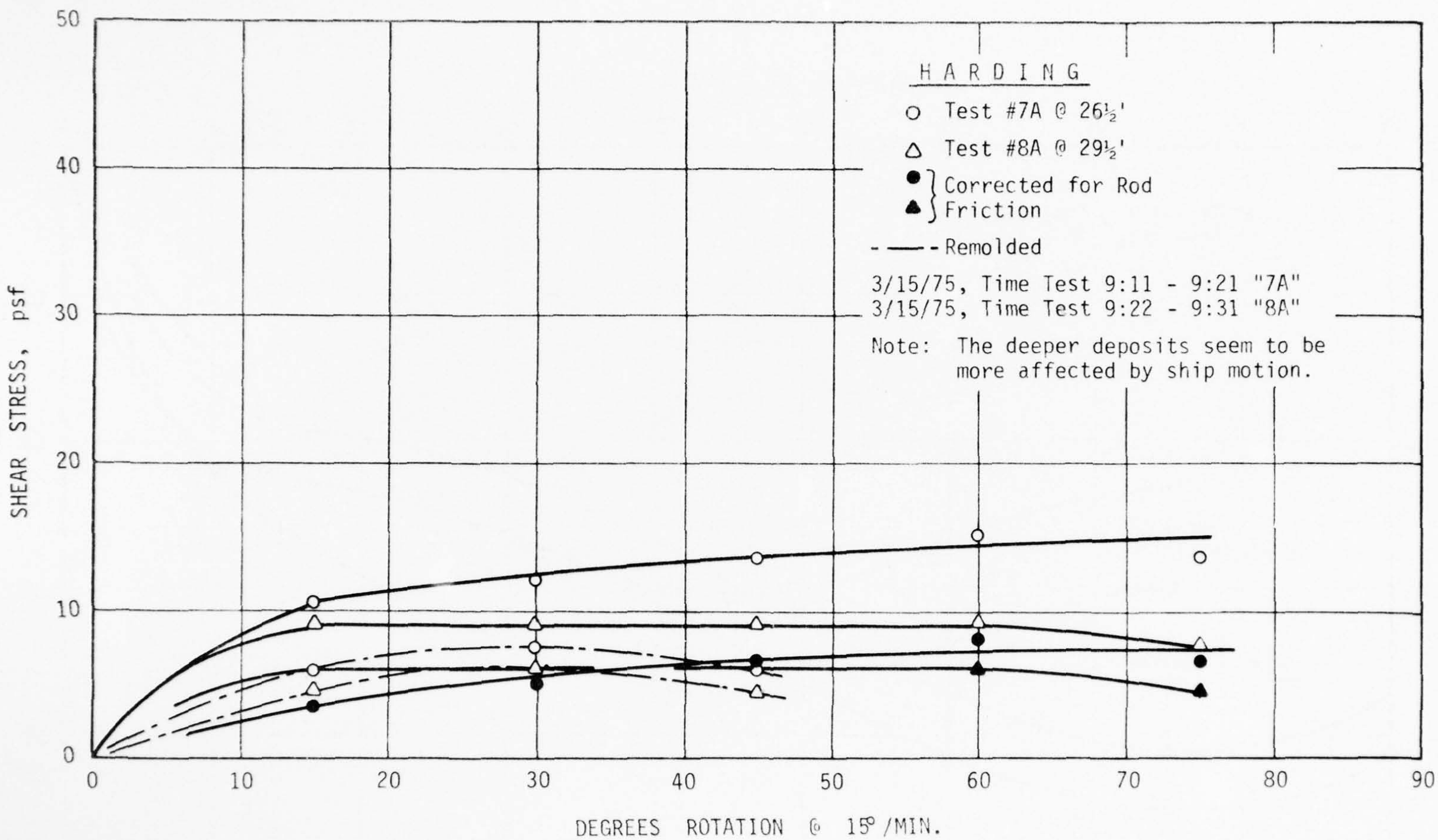


Figure 3-29. Field Vane Shear Tests - HARDING

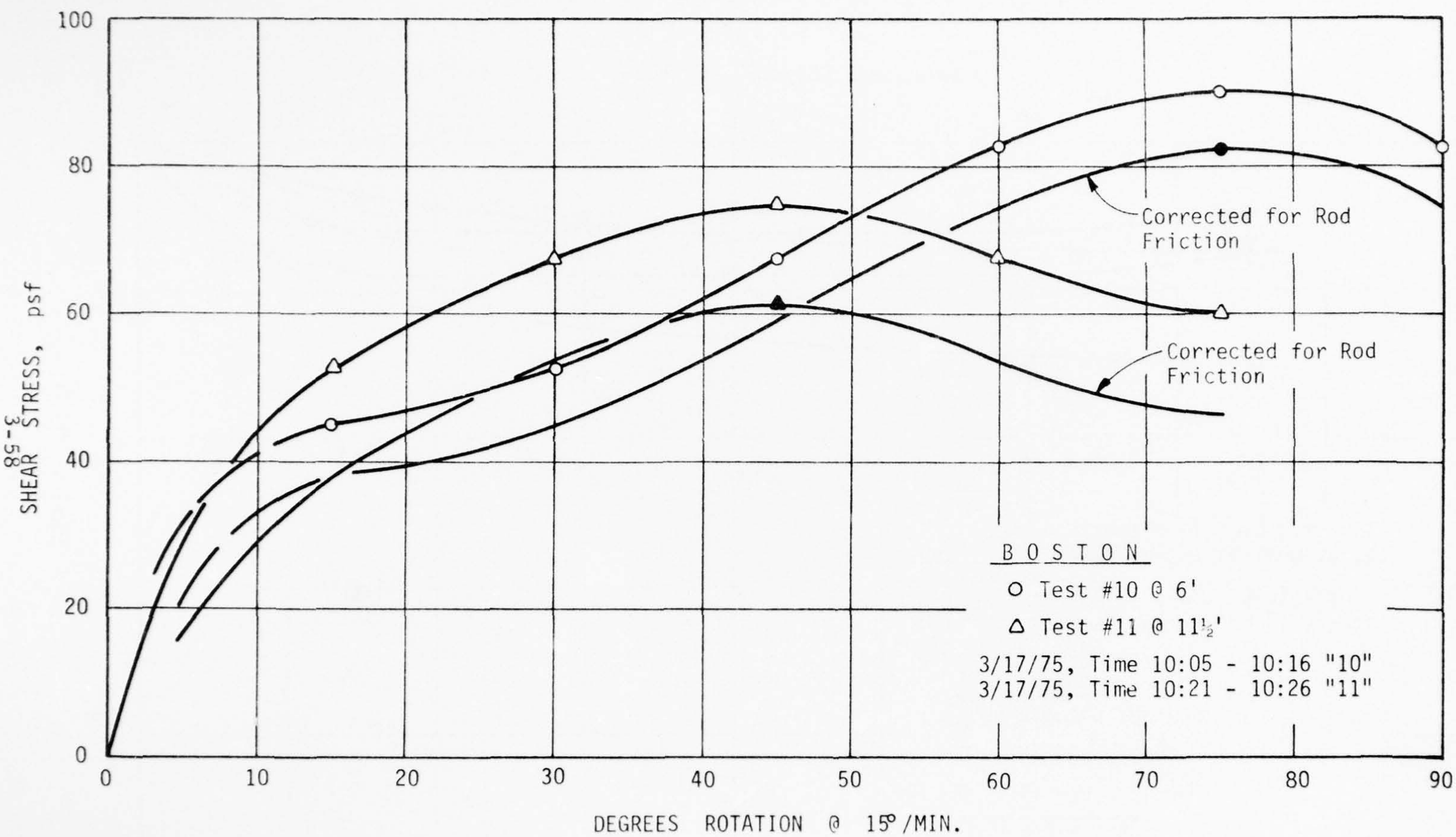


Figure 3-30. Field Vane Shear Tests - BOSTON

## TABLE 3-11

## Field Vane Shear Test Summary

| DREDGE | TEST # | DEPTH OF TEST (FT) | MAX. SHEAR STRENGTH (PSF) | SHEAR STRENGTH* CONNECTED FOR ROD FRICTION | DEGREES ROTATION | DATE & TIME OF TEST | REMARKS |
|---|---|---|---|---|---|---|---|
| HARDING** | B | 1 | 9 | 8 | 30 | 3/13/75,12:33 | 5 Gal. Bucket |
| | C | 1/2 | 10.5 | 10 | 45 | 13:04 | 1 Gal. Bucket |
| | D | 1/2 | 6 | 5 | 30 | 3/13/75,14:15 | 1 Gal. Bucket |
| | 1 | 1 | 9 | 8 | 15 | 3/14/75,11:12 | 3½"x 12" Bucket |
| | 3 | 28½ | 18 | 12 | 45 | 13:21 | |
| | 4 | 24½ | 7.5 | 7 | 30 | 14:25 | Ship at rest |
| | 5 | 29½ | 10.5 | 4 | 45 | 14:35 | Ship at rest |
| | 4A | 24½ | 7.5 | 7 | 30 | 14:49 | Ship in motion |
| | 5A | 29½ | 9 | 2 | 45 | 3/14/75,14:50 | Ship in motion (Turning) |
| | 7 | 24½ | 16.5 | 16 | 90 | 3/15/75, 8:04 | Ship at rest |
| | 8 | 29½ | 14 | 7 | 40 | 8:18 | Ship at rest |
| | 7A | 26½ | 15 | 8 | 60 | 9:11 | Ship in motion |
| | 8A | 29½ | 9 | 6 | 15 | 9:22 | Ship in motion |
| | 7B | 26½ | 15 | 12 | 45 | 10:01 | Ship in motion |
| HARDING** | 8B | 29½ | 6 | 0 | 30 | 3/15/75,10:10 | Ship in motion (Turning) |
| BOSTON | - | 4 | 15 | 10 | 30 | 3/11/75, 9:35 | 50' Forward |
| | - | 5 | 67.5 | 61 | 75 | 9:44 | 50' Forward |
| | A | 1/2 | 50 | 49 | 20 | 11:22 | Loading bucket South side |
| | B | 1 | 73 | 72 | 25 | 11:31 | Loading bucket South side |
| | B | 2 | 110 | 108 | 50 | 11:40 | Loading bucket South side |
| | C | 1 | 13.5 | 12 | 30 | 11:52 | Loading bucket North side |
| | C | 2 | 82 | 80 | 30 | 12:00 | Loading bucket North side |
| | - | 3/4 | 8 | 7 | 35 | 1:57 | 5 Gal. Bucket |
| | - | 1 | 14 | 13 | 50 | 2:21 | In barge where above sample was taken |
| | - | 10 | 10.5 | 0 | 15 | 4:05 | |
| | - | 12 | 17 | 2 | 25 | 4:26 | |
| | - | 8 | 13.5 | 3 | 30 | 3/11/75, 4:39 | |
| | 10 | 6 | 90 | 82 | 75 | 3/17/75,10:05 | |
| | 11 | 11½ | 75 | 61 | 45 | 10:21 | |
| | 13 | 1½ | 31 | 30 | 25 | 11:52 | In clam bucket |
| | 13 | 1 | 54 | 53 | 35 | 11:47 | Below clam bucket |
| | 14 | 6½ | 60 | 52 | 90 | 12:12 | |
| BOSTON | 15 | 11½ | 60 | 46 | 55 | 3/17/75,12:25 | |

*1.25psf/ft of Rod

**Rod friction assumed to take effect at 22 ft depth on 3/15/75 and 24 ft depth on 3/13/75 and 3/14/75

In the hopper dredge operation the sediment is highly disturbed when sucked into the drag head, mixed with water, and turned into a slurry while being pumped into the hoppers. In this state the soil, at a water content generally above its liquid limit, behaves as a highly viscous fluid rather than a plastic material. The liquid limit (LL) of a soil is the water content, expressed as a percentage of the dry weight of the solids, at which the soil changes from a plastic form to a liquid form (see ASTM D653-67). This point is generally considered as the borderline between soil and fluid mechanics, and it is generally accepted that the shear strength, as it is understood in soil mechanics terms, becomes zero in soils with a water content above their liquid limits. At that point the viscosity of the fluid is then considered as a type of shear strength, but as measured in terms of fluid mechanics. The results presented from the hopper dredge tests tend to bear out the fact that the shear strength of the material in the hopper dredge is at or near zero, with all of the values above zero probably falling within the range of accuracy of the measuring equipment.

In the clamshell dredge operation the sediment is dumped into the barge pockets with a clamshell which has been dropped into the bay bottom sediments until full. The material in this case, although disturbed, is deposited in clumps, some of which retain their soil consistency and therefore retain measurable shear strength.

The fluctuations noted in the results of the shear strength measurements on the BOSTON are due to this random clumping of the dredged material, which results in water-filled voids between the clumps, and therefore in extensive stratification of the deposit within a given cross section of the barge pockets.

As can be noted on Figures 3-29 and 3-30, an allowance for rod friction was made in calculating the final values of shear strength; the reason being that the torque measured at the level of the ships' decks included not only the resistance of the dredged material to the shear vane, but also the friction of the extension rods. In order to determine the value of this rod friction a series of tests was made in which the rods were turned without attaching the shear vane. The torque values were then used in calculating shear strength in the ordinary way; based on an average, a constant of rod friction resistance per foot of rod penetration amounting to 1.25 psf of shear strength was calculated. The data are presented in graphical form in Figure 3-31 (see Appendix B for a detailed discussion of assumptions involved).

Penetration Tests - To determine the bearing capacity of the dredged materials, penetration tests were made using flat steel plates, one a 6 in. dia. circle and the other a 12 in. square, attached to drill rods and weights. Figure 3-32 presents the graphical results and these are summarized in Table 3-12. The zero penetration line on the bottom of the graph represents the level of the dredged materials at which resistance to the plate penetration was first encountered, i.e., where settlement of the plate/rod system first was anything less than just sinking through water. Since in no case was sufficient time available to balance the weight on the plates in such a way that settlement of the plates could be arrested, it was not possible to calculate any reliable bearing capacity values, based on plate penetration tests. The results for the 12-inch square plate do, however, indicate that about two feet into the solids the soil becomes relatively stronger, i.e., the settlement slows down to a rate of about 1 per minute under a 30 to 40-pound weight. This would indicate that the bearing capacity of the material at that point would be

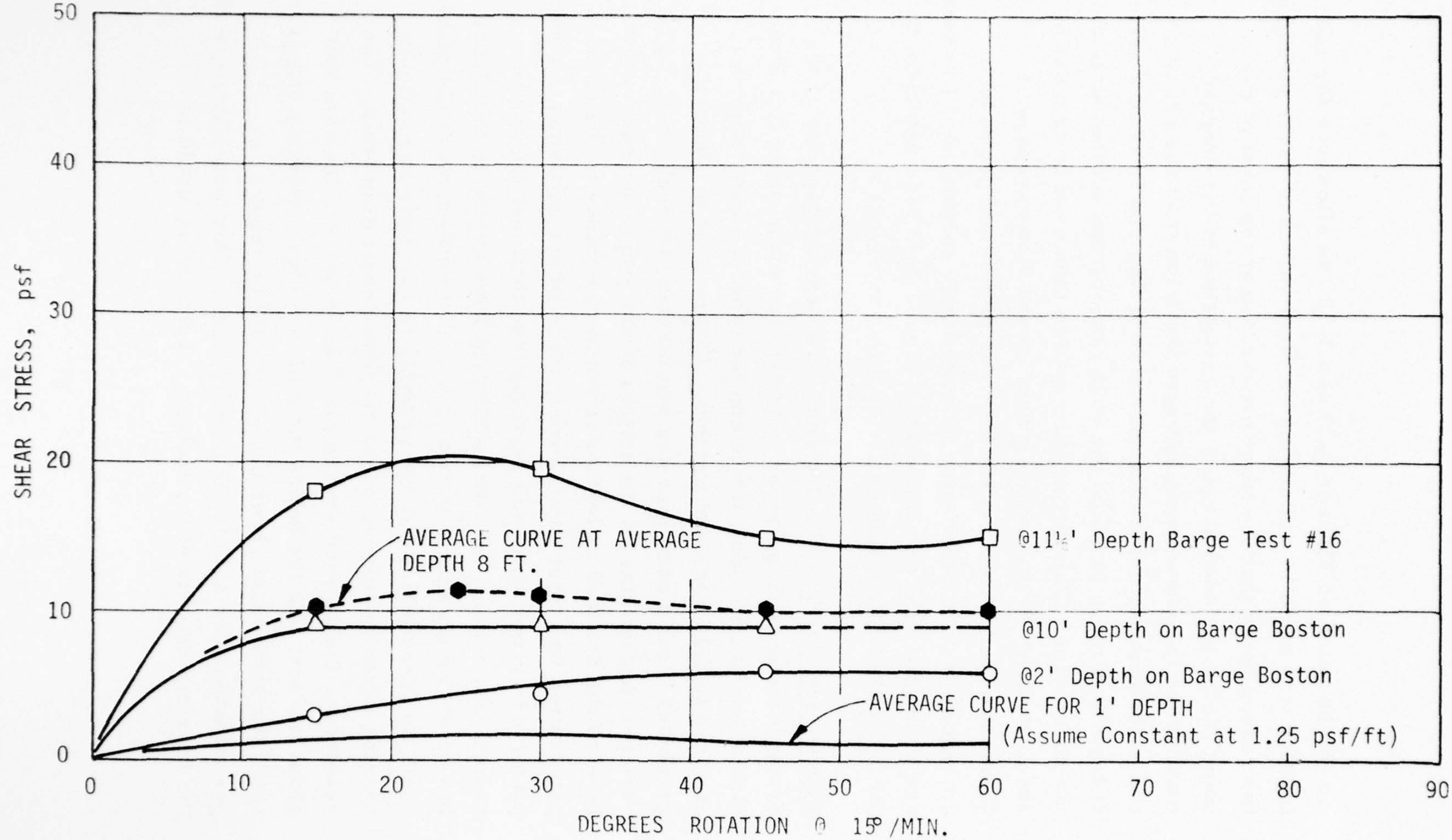


Figure 3-31. Rod Friction Data for Field Vane Shear Tests

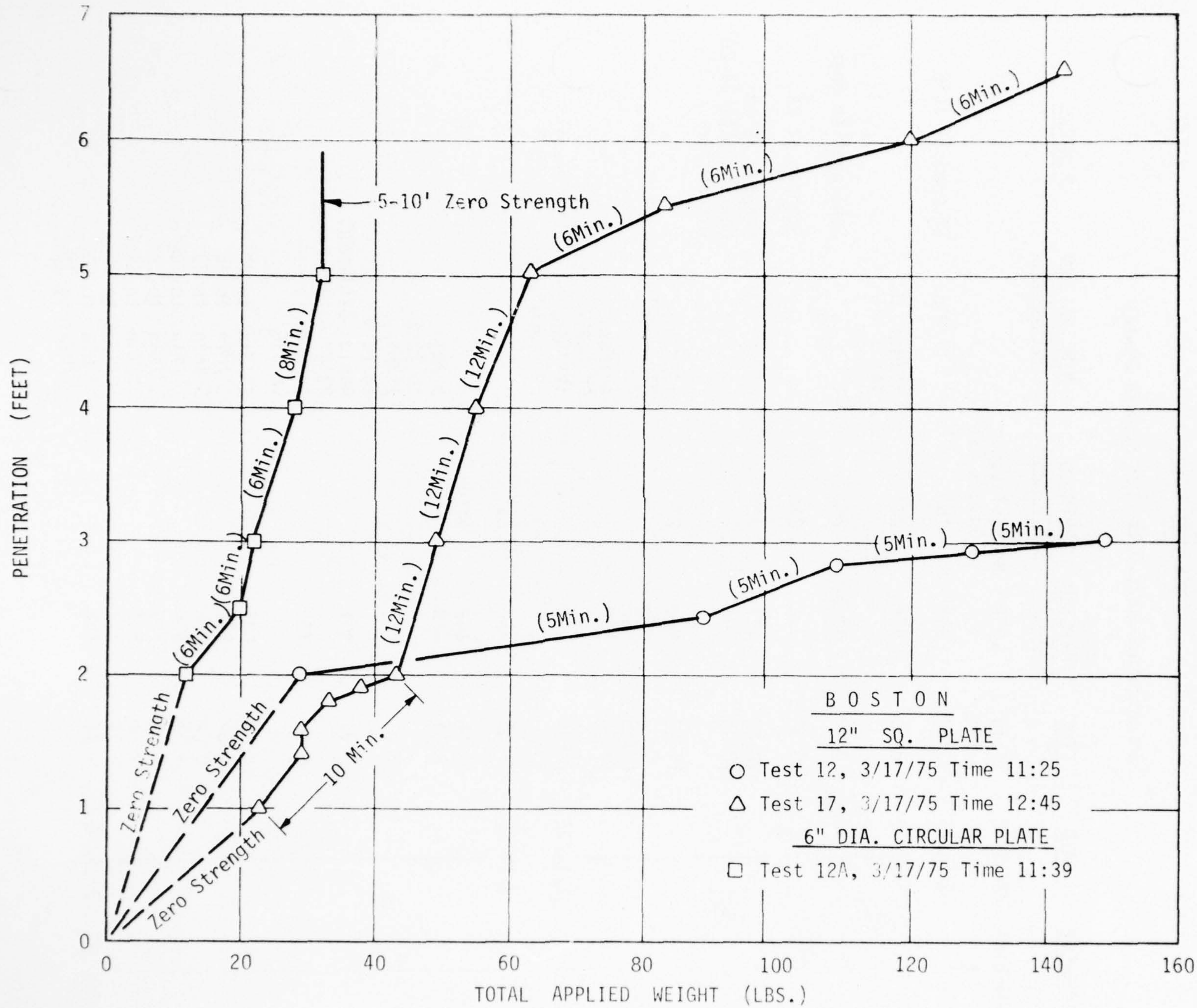


Figure 3-32. Penetration Tests

TABLE 3-12

Penetration Test Data (12" Square Plate)

| DREDGE | DATE | TIME | APPLIED WEIGHT | DEPTH TO RESISTANCE | MIN. RATE OF SETTLEMENT | REMARKS |
|---|---|---|---|---|---|---|
| HARDING | 3/13/75 | 14:10 | 100# | No resistance to bottom (31') | | |
| | 3/13/75 | 15:00 | 30# | 16' | 1"/Min. | Stopped at 16'5" |
| | 3/14/75 | 8:55 | 57# | 24' | 1"/Min. @ 28' - 31½' | |
| | | 9:10 | 43.5# | 28' | | Continued to move |
| | | 10:30 | 36# | 24' | | Stopped at 24' |
| | | 10:45 | 44# | 23' | | Stopped at 23' |
| | | 10:45 | 50# | | | Continued to 27-28' out of time |
| | | 12:07 | 44# | 27' | 1"/Min. 27½ - 29' | |
| | | 12:15 | 65# | | | |
| | | 12:20 | 90# | | 4"/Min. 28 - 29' | |
| | 3/14/75 | 12:20 | 90# | | 12"/Min. 29 - 31½' | |
| | 3/15/75 | 7:55 | 44# | 21' | | |
| | | 8:26 | 44# | 21'9" | | |
| | | 8:31 | 66# | | 5"/Min. to 22'2" | |
| | | 8:31 | 66# | | 1"/Min. to 22'4" | |
| | | 8:31 | 66# | | Rapid settlement 22'4" - 28' | |
| | | 8:40 | 74# | | 2"/Sec. to 31½' | |
| | | 9:34 | 53# | 20' | 4"/Min. to 22' | |
| | | 9:34 | 53# | | 1'/Min. to 25' | |
| | | 9:34 | 53# | | 5"/Min. to 26' | |
| | | 9:34 | 53# | | 1'/Min. to 26½' | |
| | | 9:34 | 53# | | 5"/Min. to 28' | |
| | | 9:34 | 53# | | 1"/Min. to 29¼' | |
| | | 9:44 | 74# | | 1"/Sec. to 30¼' | |
| HARDING | 3/15/75 | 9:44 | 74# | | 2"/Min. to 30½' ½"/Min. to 31½' | |

sor what less than 30 psf (the bearing area of the plate being one square foot).

One plate bearing test was run in conjunction with the sinking sphere tests during which a total of 26.6 pounds was supported on the 12-inch square plate without significant settlement past a depth of 21.7 feet in hopper No. 3 on the HARDING. Assuming some weight reduction due to buoyancy, the actual estimated bearing capacity in this test was about 25 psf. This value agrees well with the assumptions made for the penetration tests above and also agrees fairly well with results obtained during the sinking sphere tests to be discussed below.

Sinking Sphere Tests - The sinking sphere tests were performed using a lead sphere which was lowered on a line into the dredged material. For the purpose of relating the results to bearing capacities, the following assumptions were made: 1) the weight of the sphere minus the weight of the displaced mud equaled 1.5 pounds; 2) the surface area of the bottom hemisphere was assumed as the involved bearing area. The depths below water at which the sphere first came to rest ranged from about 16.5 to 27.5 feet. Typical run results are shown in Figure 3-33. These depths were comparable to the depths at which resistance during the plate penetration tests was encountered. When the sphere came to rest, the force needed to support it could then be considered a bearing capacity, which equaled about 34 psf (the weight of 1.5 pounds divided by the area of the hemisphere of 0.044 square feet). As in the plate bearing tests, the value here also represents an ultimate bearing capacity just on the verge of failure. It again fits in well with data from the penetration tests. Also when one considers the ultimate bearing capacity to be equal to roughly five times the shear strength, an accepted rule of thumb in soil mechanics, the 25 to 35 psf figure for bearing capacity fits very well with data recorded during the field vane shear tests aboard the HARDING.

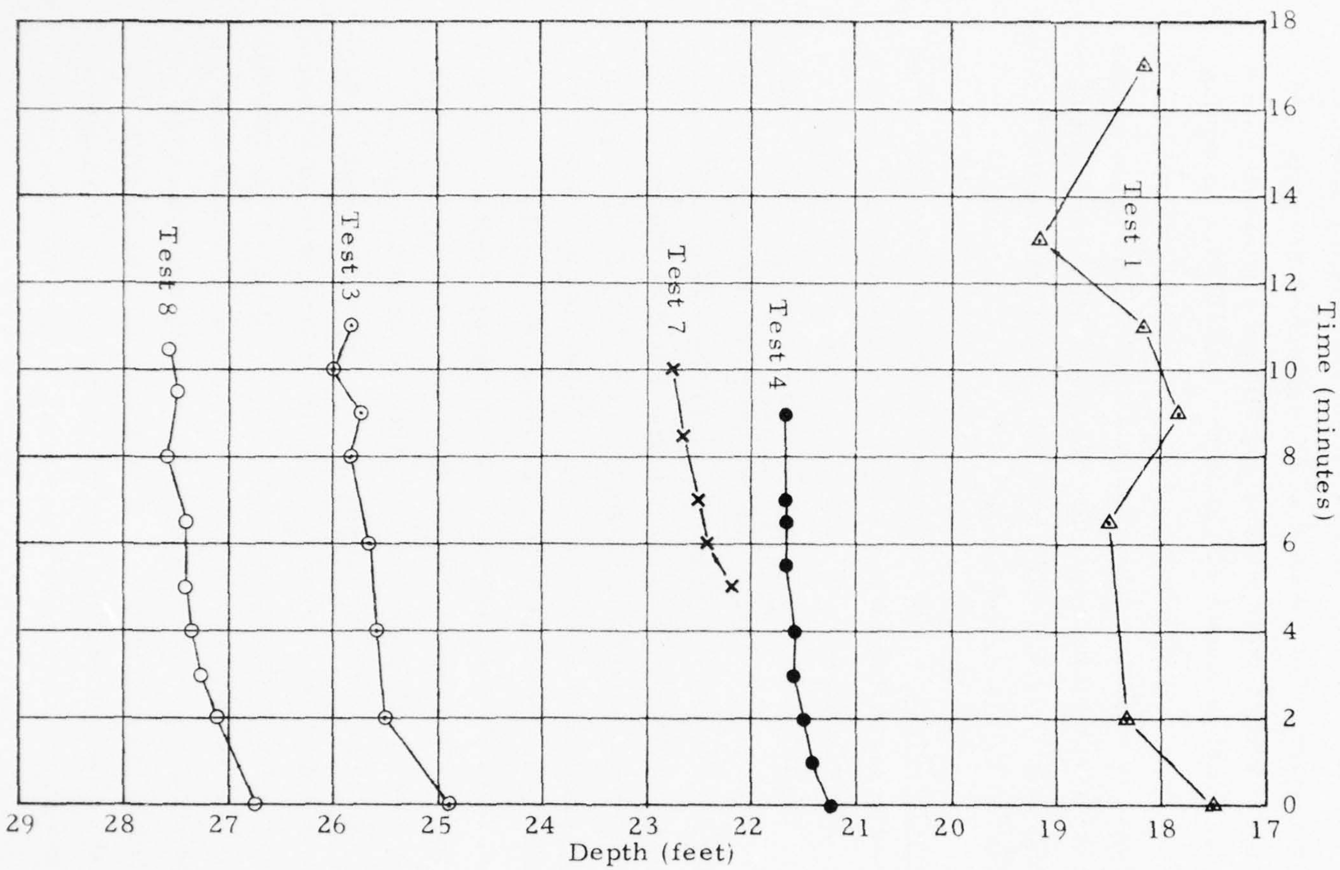


Figure 3-33. Sinking Sphere Tests on the HARDING

An interesting phenomena was noted on some runs using the sinking sphere. Figures 3-34 show the variation in the depth in the hopper at which the sphere was supported. In Test 1 large variations in this surface location were observed as the vessel maneuvered. A turn to port caused the sphere depth to decrease on the starboard side and increase on the port side, indicating that the fluid was shifting as the vessel heeled. Variations as great as 14 inches were observed on turns. There seems to also be a trend toward the sphere coming to equilibrium at a deeper depth in the hopper each time a maneuver takes place, suggesting that settling is taking place as a result of the maneuver. Swirls were evident on the surface of the hoppers during the maneuvers which suggests that pockets of water may be interspersed throughout the hopper, creating large void spaces that surface as the load shifts. If this hypothesis is correct, it may be possible to relieve these pockets by maneuvering the vessel, thus increasing the storage volume (if overflow is allowed) and increasing the average percent solids in the lower parts of the hoppers. Such maneuvers would aid in maximizing the total load, but might not necessarily achieve the most economic loading.

Figure 3-35 shows the results of a special run in which the dredge was filled in the conventional manner and then, rather than transiting to the dumpsite, it hove-to for 15 minutes and then transited to the dump site. The shape of the curve while hove-to for 15 minutes is quite similar to the shape of curves for other runs in which the dredge proceeded immediately to the dump site. This suggests that the behavior of the sphere, and the material supporting it, is the same whether transiting or not. Thus, vibrations do not appear to significantly affect the bearing strength of the material at the depth indicated during the 15 minute time interval. Settling of the material is once again evident as the dredge turned and maneuvered.

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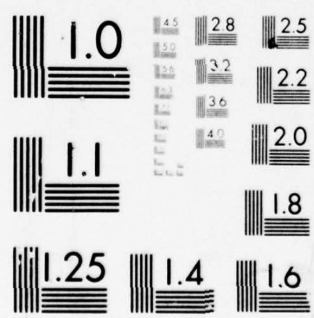
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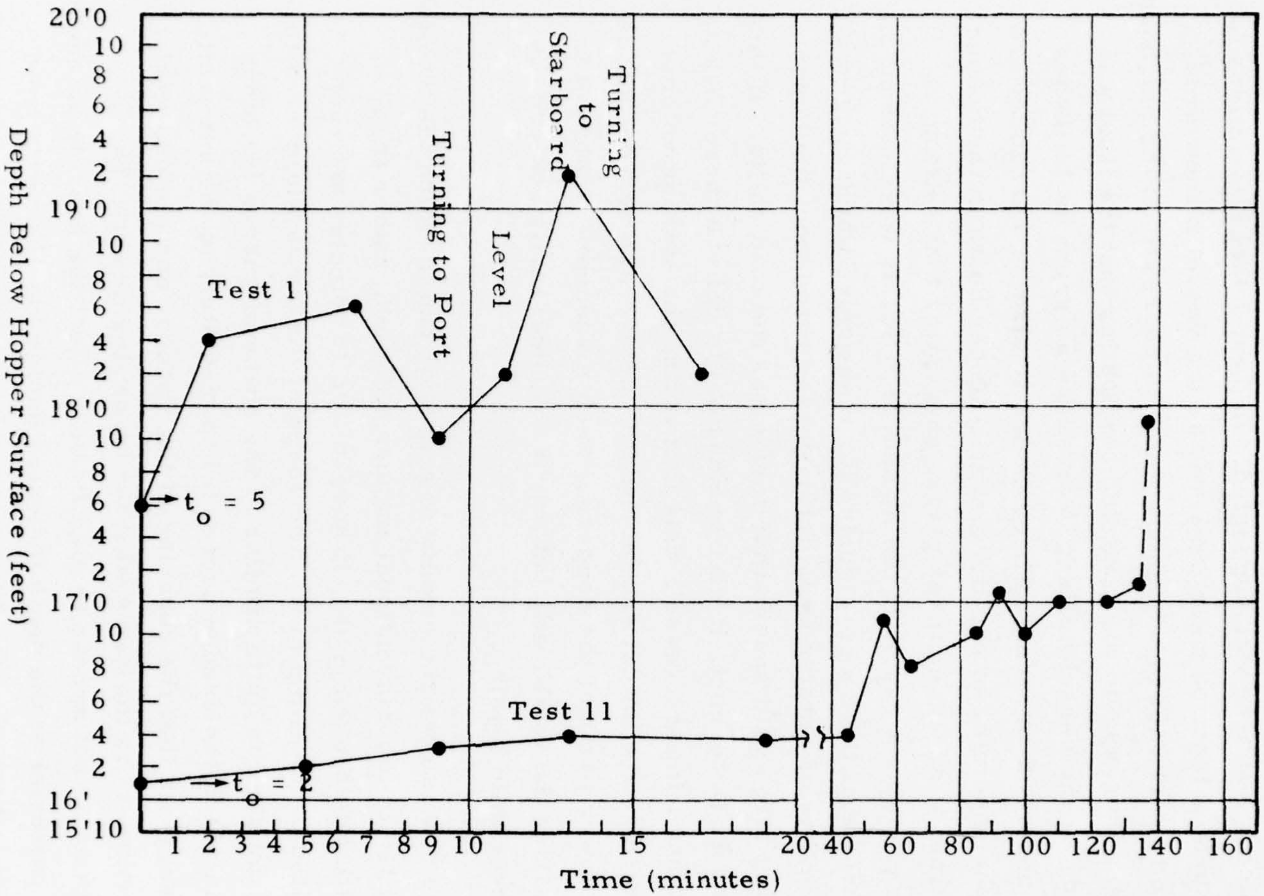



Figure 3-34. Sinking Sphere Tests Indicating Fluid Motion in Hoppers

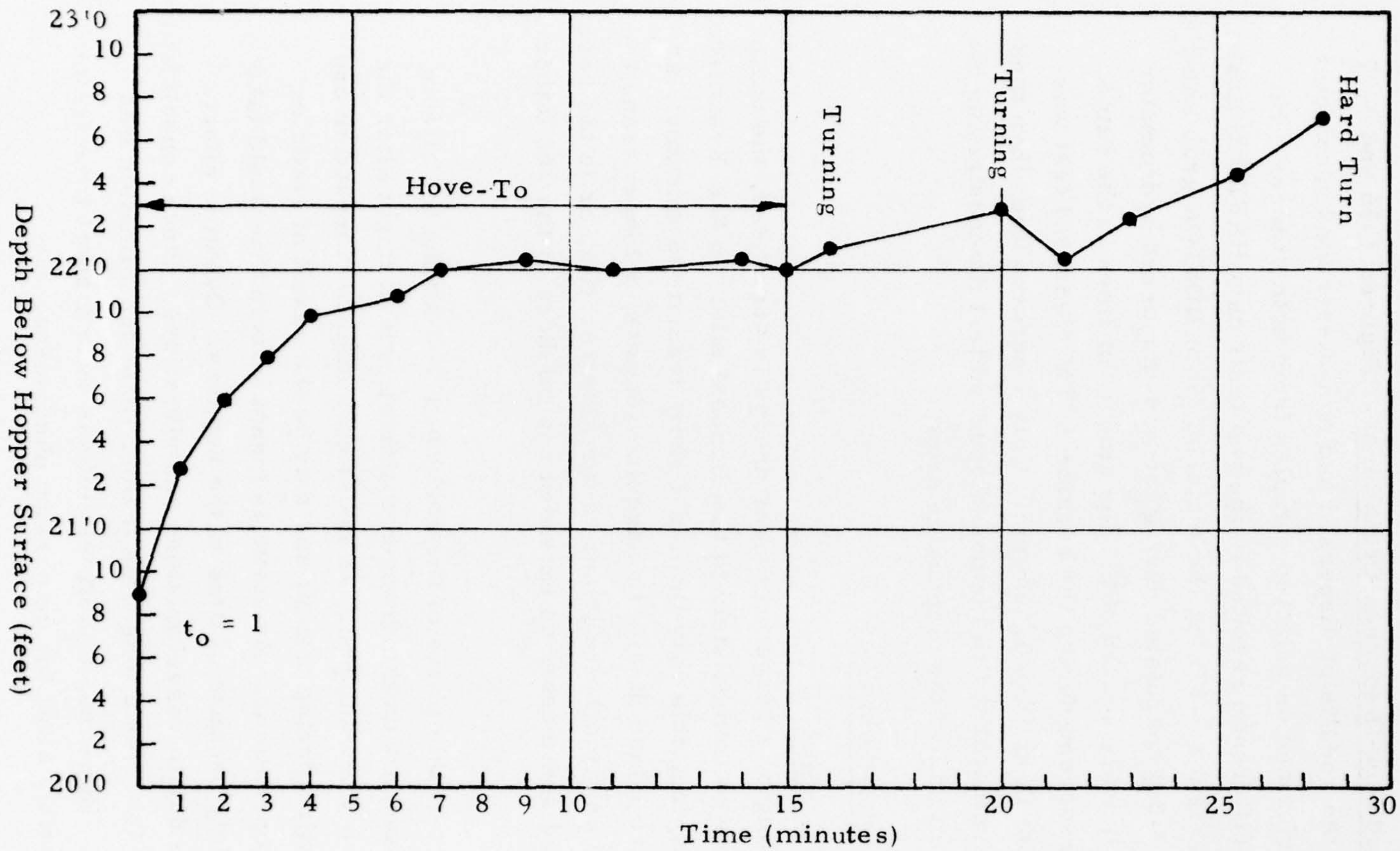



Figure 3-35. Sinking Sphere Tests Indicating Hove-To versus Transiting Conditions

Dispersed Non-Dispersed Grain Size - Figures 3-36 and 3-37 present the results of dispersed and non-dispersed hydrometer tests performed on selected samples from both dredges. No marked difference resulted in the two tests on a HARDING grab sample (Figure 3-37) but the material from BOSTON grab sample (Figure 3-36) coagulated during the non-dispersed hydrometer test and quickly settled out. The amount of fines in the same sample recorded during the regular ASTM dispersed test was 60 percent finer than 0.005 mm. Only 7 percent finer than these sizes were recorded in the non-dispersed test demonstrating the obvious effects of the dispersing agent.

## D. Transport Phase

### 1. General

After filling of a barge or hopper dredge is completed, the dump vessel moves immediately to the disposal site. In San Francisco Bay dredging, this may involve a short transit (15 minutes), as from Mare Island Straits to Carquinez Straits, a longer transit (2 hours) as from Mare Island to the Alcatraz site, or in the case of polluted sediments, an extended run offshore to the 100 fathom line.

The intent of this phase of the study was to examine the effects, due to transport on the dredged material, which might alter the dispersion when dumped. These effects include compaction and consolidation during transit and may be the result of static or dynamic operations. By static is meant effects that would take place solely because of time in the hoppers. Dynamic effects are those due to vessel motions and vibrations. Since consolidation normally involves a long time frame, little consolidation was expected. However, thixotropic effects over a short time frame could possibly alter the dispersion phenomena.

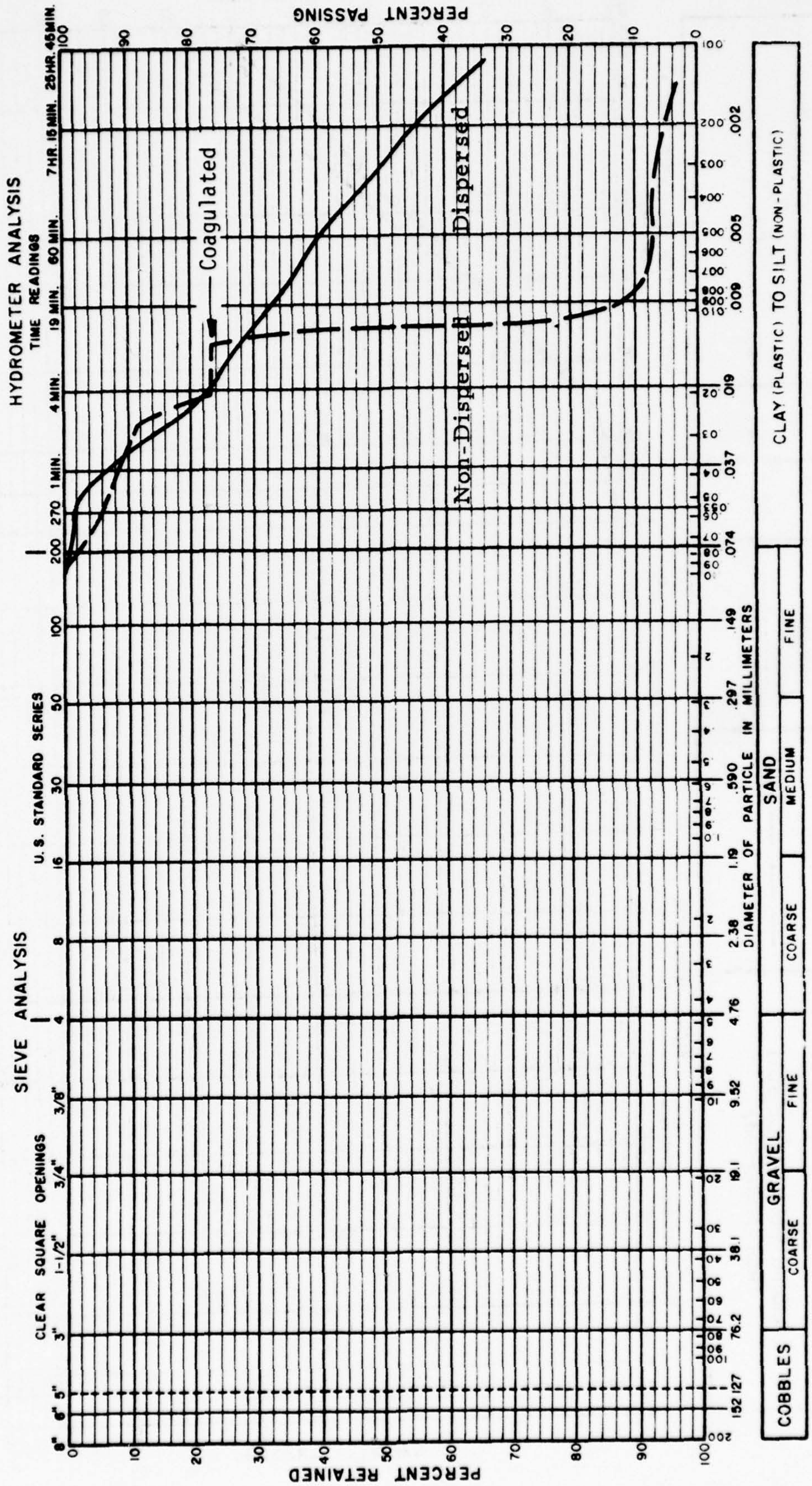



Figure 3-36. Dispersed & Non-Dispersed Grain Size Analyses - BOSTON

| SAMPLE NO. | SYMBOL | DEPTH | LL | PI | UNIFIED CLASSIFICATION |
|---|---|---|---|---|---|
| HARDING GRAB SAMPLE #081 (Dispersed) | ———— | —— | 82 | 51 | CH |
| HARDING GRAB SAMPLE #081 (Non Dispersed) | — — — — | —— | 82 | 51 | CH |

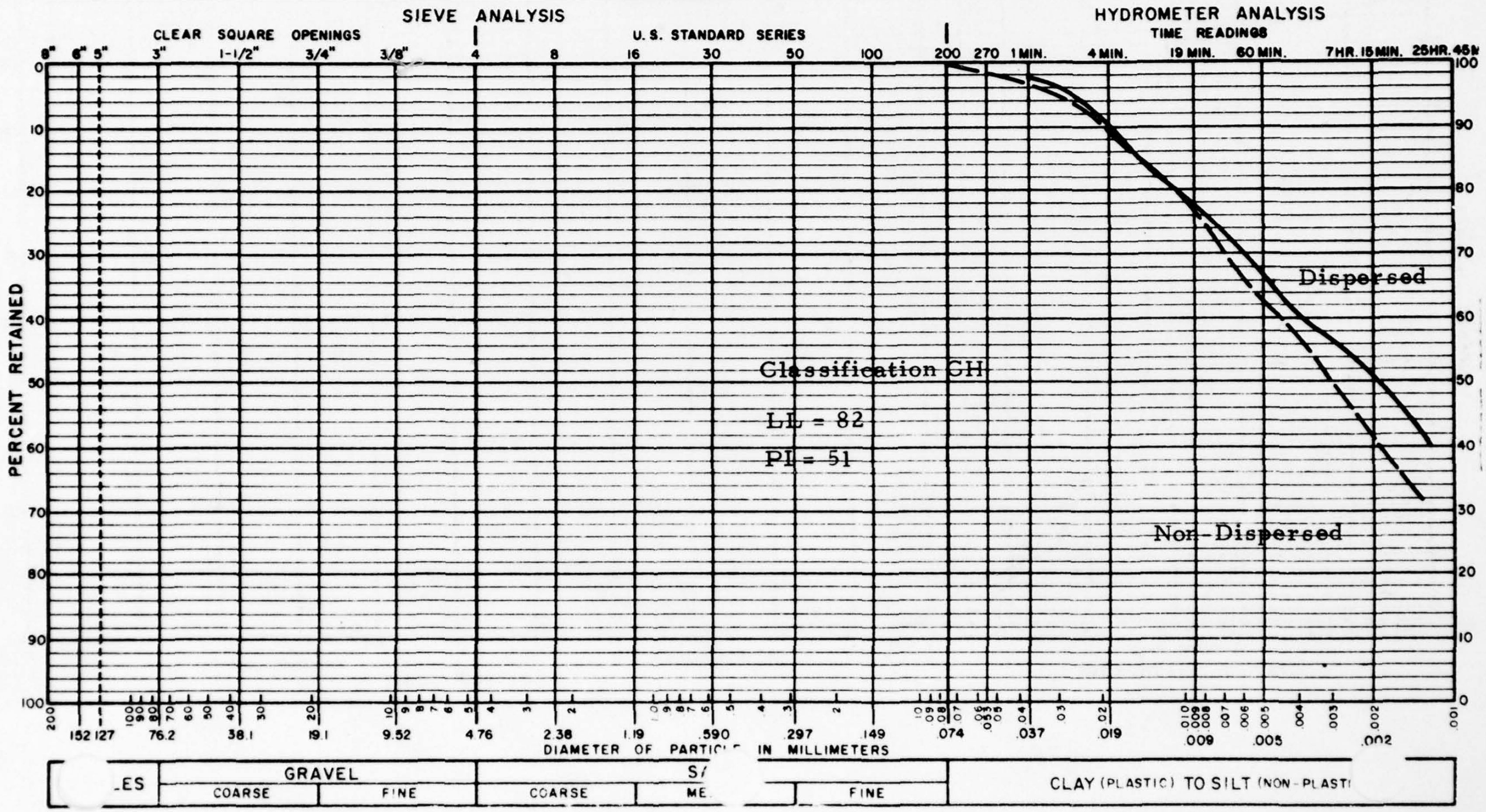



Figure 3-37. Dispersed & Non-Dispersed Grain Size Analyses - HARDING Grab Sample #081

The approach used was to conduct field measurements on the hopper dredge HARDING, and then laboratory simulations of both the static and dynamic conditions for the range of physical characteristics observed in the field. Dredged materials were a silty material from Pinole Shoal and a clay-like material from Mare Island Straits. Measurements of oxygen demand and moisture content were reported under the dredging phase section. In this section, vibrations and their effect on the dredged material are discussed.

## 2. Vibration Measurements on the HARDING

### Measurements

It has been observed in the past that dredged material characteristics seem to change during transit to the dump site. Measurements were made to describe the vibration environment on the HARDING, so that a series of controlled laboratory experiments could be conducted to measure the significance of this effect, if any.

Harmonic motions were recorded for the forward section of the hopper with a vertically mounted adjustable probe at depths of 5 feet 4 inches, 12 feet 6 inches, 17 feet, 17 feet 6 inches, and 25 feet below the fluid surface level of a full hopper. Vibration measurements of the hopper structure adjacent to the probe were also recorded. Data were recorded during filling, transiting and dumping.

Measurements of the harmonic motion of the water and sediment within the forward section of the No. 3 hopper were made by positioning a special probe (see Appendix D) containing three piezoelectric velocity transducers and three linear accelerometers in the fluid at several depths. Measurements were also made of

the vibrations on the hopper side adjacent to the probe using three additional velocity transducers. Figure 3-38 shows a diagram of the instrumentation system used.

Vibration measurements were made with Bell and Howell Type 4-155-001 piezoelectric velocity transducers. Within the probe the velocity transducers were mounted to record vertical, transverse, and fore and aft harmonic motion. Three velocity pickups for measuring the hopper side vertical, transverse, and fore and aft vibrations were assembled as a movable triad having a magnetic base for attaching to the hopper structure.

The three accelerometers mounted within the probe for measuring vertical, transverse, and fore and aft accelerations were Setra Systems Model 117 High Output Linear Units. These units were designed for vertical mounting face-up. Since in no case were these static conditions met, the steady state signal (g load) was of such magnitude that only low gain could be used. These low gains generally produced too low a recorded signal to be observed.

The conditioning boxes contained the power supplies for the transducers and in addition the vibration box provided filtering of the vibration transducer output. Transducer output signals passed through the conditioning boxes to a switch box and then to a Neff Type 123 wideband, differential DC Amplifier designed for high accuracy amplification of low level signals. The amplifier package consisted of eight individual amplifiers; two spares and one for each of six data channels. The solid state amplififiers have step-gain settings ranging from 10 to 1000. Output from each amplifier was recorded on a six channel Brush Mark 260 Ink-type Oscillograph. The Brush Mark 260 Recorder also has four event channels. One of these was used to record accurate 5 cps timing

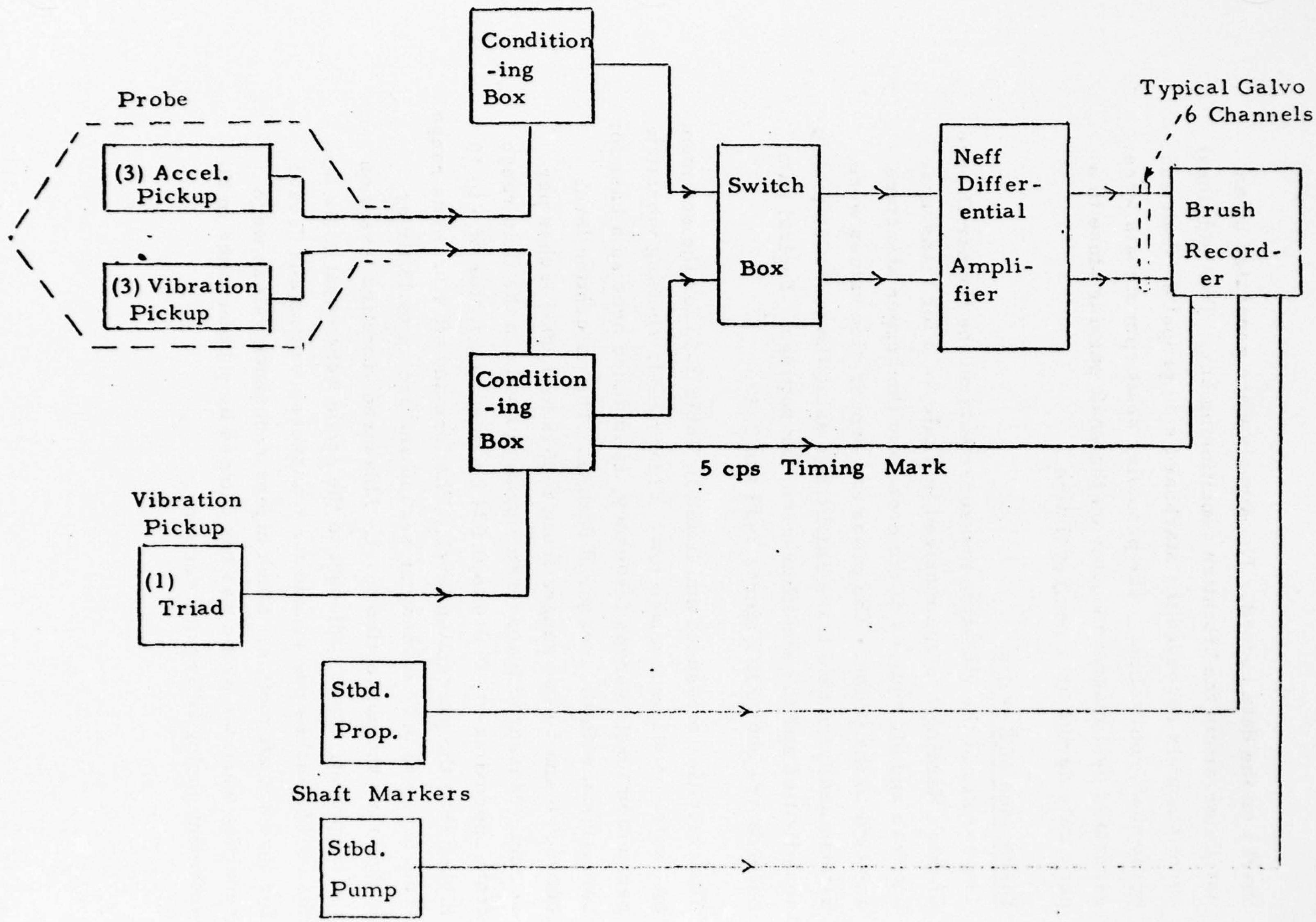



Figure 3-38. Diagram of Instrumentation to Measure Vibrations

marks on the data record. The signals were generated by an oscillator within the vibration conditioning box. Two additional event channels recorded the starboard side propeller shaft and pump shaft rpm values. The propeller shaft rpm signals were generated by a magnet mounted on the shaft passing close to a proximity device once per revolution.

Discussion of Results

The results of the vibration measurements on the Hopper Dredge Chester Harding are summarized in Table 3-13 for transverse and fore and aft motions of the probe and the hopper structure adjacent to the probe. The probe and hopper side values were determined from data trace amplitudes using the vibration pickup sensitivities and the amplifier-recorder settings. Typical data records are shown in Figures 3-39 and 3-40.

The velocities measured and listed in Table 3-13 for the sediment and water of the hopper are low. Figure 3-41, showing vibration perception levels versus frequency, is included here as a base for comparison with the measured results. The perception level between 6 and 50 cps ranges from 0.0018 to 0.0036 inches per second. Measured transverse vibration levels in the fluid range from approximately 0.014 to 0.034 or roughly a factor of 5 to 10 higher than the perception level. The fore and aft vibrations range from 0.024 to 0.05 inches per second and are up to 15 times higher than the perception level. Measured vibration levels on the hopper structures adjacent to the probe were equal to or less than 0.056 inches per second for transverse motion and 0.0135 for fore and aft motion. Much higher resonance values were recorded when the probe was not damped by submergence in the sediment during filling and dumping.

TABI -13

HOPPER DREDGE CHESTER HARDING

MEASURED VIBRATIONS WITHIN HOPPER AND ON ADJACENT HOPPER SIDE

| Measurement | RPM | Transverse Vibr., In/sec. Probe | Fluid (Mud) | Order/Freq. | Fore & Aft Vibr., In/sec. Probe | Fluid (Mud) | Order/Fr |
|---|---|---|---|---|---|---|---|
| 5' 4" DEPTH - MARCH 14 | | | | | | | |
| Probe - Not Covered - Filling | Prop. 95 Pump 260 | 0.0441 | - | 7.4 cps (Resonance) | 0.0267 | - | 52 cps |
| Probe - Covered Partially Full | 95 260 | 0.0062 | 0.0092 | 26 cps | 0.0110 | 0.0090 | 13 cps |
| Probe - Covered Full | 97 262 | 0.0078 | 0.0116 | 25, 58 cps | 0.0152 | 0.0125 | 8th (13 cps |
| Probe - Full In Transit | 188 | 0.0114 0.0023 | Indeter. Resonance 0.0038 | 2nd (6.3 cps) 59 cps | 0.0193 | 0.0286 | 26 cps |
| Hopper Side - Full | 188 | 0.0463* | - | 8th (25 cps) | 0.0111* | - | 8th(25 cps) |
| 12' 6" DEPTH - MARCH 14 | | | | | | | |
| Probe - Not Covered-Filling | Prop. 95 Pump 260 | 0.658 0.031 | - - | 1.2 cps(Reson) 15.5 cps | 0.0851 0.0124 | - - | 1.2cps(Res 30 cps |
| Hopper Side -Probe Not Covered | 95 260 | 0.0247* | - | 88 cps | 0.0056* | - | 8th & Misc. |

*Vibration on Hopper Side

(Continued on following pages

TABLE 3-13 (cont.)

HOPPER DREDGE CHESTER HARDING

MEASURED VIBRATIONS WITHIN HOPPER AND ON ADJACENT HOPPER SIDE

| Measurement | RPM | Transverse Vibr., In/sec. Probe | Transverse Vibr., In/sec. Fluid (Mud) | Transverse Vibr., In/sec. Order/Freq. | Fore & Aft Vibr., In/sec. Probe | Fore & Aft Vibr., In/sec. Fluid (Mud) | Fore & Aft Vibr., In/sec. Order/Fre |
|---|---|---|---|---|---|---|---|
| **12' 6" DEPTH - MARCH 14 (Continued)** | | | | | | | |
| Hopper Side - Probe Covered-Not Full | 95<br>260 | 0.0247* | - | Approx. 46 cps | 0.0097* | - | Mixed |
| Probe - Covered Partially Full | Prop. 96<br>Pump 261 | 0.0078 | 0.0132 | 8th (12.7 cps) | 0.0097<br>0.0055 | 0.0165<br>0.0093 | 21 cps<br>151 cps |
| Probe - Full | 96 | 0.0054<br>0.0023 | 0.0091<br>0.0039 | 8th (12.8 cps)<br>61 cps | 0.0034<br>0.0028 | 0.0058<br>0.0048 | 8th (12.8c]<br>156 cps |
| Hopper Side Full | 96 | 0.0148*<br>0.0074* | -<br>- | 8th (12.7 cps)<br>86 cps | 0.0069* | - | 8th (12.7 c |
| Probe Dumping | | 0.879<br>0.0465 | -<br>- | 1.2 cps (Reson)<br>10th (15.8 cps) | 0.169<br>0.207<br>0.0104 | -<br>-<br>- | 1.2cps(Res<br>22.5 cp<br>158 cps |
| **17' 6" DEPTH - MARCH 14** | | | | | | | |
| Probe - Not Covered-Filling | Prop. 95<br>Pump 260 | 0.800<br>0.126<br>0.031 | -<br>-<br>- | 0.7 cps (Reson)<br>4th (6.3 cps)<br>38 cps | 0.1189<br>0.0207 | -<br>- | 0.7cps(Res<br>Appx.50cps |

*Vibration on Hopper Side

TABLE 3-13 (Cont.)

HOPPER DREDGE CHESTER HARDING

MEASURED VIBRATIONS WITHIN HOPPER AND ON ADJACENT HOPPER SIDE

| Measurement | RPM | Transverse Vibr., In/sec. Probe | Fluid(Mud) | Order/Freq. | Fore & Aft Vibr., In/sec. Probe | Fluid(Mud) | Order/Freq. |
|---|---|---|---|---|---|---|---|
| **17' 6" DEPTH - MARCH 14 (Continued)** | | | | | | | |
| Probe - 1st Covered | Prop. 95<br>Pump 260 | 0.0620 | 0.1056 | 17.4 cps | 0.0276<br>0.0083 | 0.0470<br>0.0141 | 25 cps<br>121 cps |
| Hopper Side-Probe Covered - Filling | 94<br>260 | 0.0154* | - | 58 cps | 0.0069* | - | 12.6 cps |
| Probe - Full Pumping | 94<br>260 | 0.0058 | 0.0099 | 8th (12.6 cps) | 0.0138 | 0.0235 | 124 cps |
| Hopper Side Full - Pumping | 95<br>260 | 0.0043* | - | 12.6 cps | 0.0083* | - | 124 cps |
| Probe Dumping | | 0.963<br>0.237 | -<br>- | 0.62 cps (Reson)<br>6.3 cps | 0.1069<br>0.0414 | -<br>- | 0.62cps(Res)<br>Mixed |
| **25' DEPTH - MARCH 14** | | | | | | | |
| Probe - Covered Filling | Prop. 118<br>Pump 267 | 0.110 | 0.1870 | 8 cps | 0.0200 | 0.0340 | 100 cps |

*Vibration on Hopper Side

TABLE 3-13 (cont.)

HOPPER DREDGE CHESTER HARDING

MEASURED VIBRATIONS WITHIN HOPPER AND ON ADJACENT HOPPER SIDE

| Measurement | RPM | Transverse Vibr., In/sec. | | | Fore & Aft Vibr., In/sec. | | |
|---|---|---|---|---|---|---|---|
| | | Probe | Fluid(Mud) | Order/Freq. | Probe | Fluid(Mud) | order/Freq. |
| 25' DEPTH - MARCH 14 (Continued) | | | | | | | |
| Probe - Covered Partially Full | Prop. 114 Pump 259 | 0.0273 | 0.0465 | 8 cps | 0.0241 | 0.0411 | 100 cps |
| Hopper Bracket Partially Full | 114 259 | 0.0180* | - | 32 cps | 0.0099* | - | 39 cps Some 8th |
| Probe - Full Pumping | 95 258 | 0.0109 | 0.0186 | 39 cps | 0.0124 | 0.0211 | 38 cps |
| Hopper Bracket Full | 118 | 0.0174* | - | 8th (15.8 cps) | 0.0086* | - | 37 cps |
| Probe - Full In Transit | 189 | 0.0202 | 0.0344 | 8th (25 cps) | 0.0152 | 0.0259 | 15.7 cps |

*Vibration on Hopper Bracket

TABLE 3-1( ont.)

HOPPER DREDGE CHESTER HARDING

MEASURED VIBRATION LEVELS WITHIN HOPPER AND ON ADJACENT HOPPER SIDE

Fwd Section of Aft Hopper — Data of March 14 and March 15, 1975

*Vibration on Hopper Side

| Measurement | RPM | Transverse Vibr., In/sec. Probe | Transverse Fluid (Mud) | Transverse Order/Freq. | Fore & Aft Vibr., In/sec. Probe | Fore & Aft Fluid (Mud) | Fore & Aft Order/Freq. |
|---|---|---|---|---|---|---|---|
| **17' DEPTH - MARCH 15** | | | | | | | |
| On Hopper Side (Pumping) | Prop. 117<br>Pump 260 | 0.0309* | - | 16.6 cps | 0.0104* | - | 15.5 cps |
| In Probe (Pumping) | 125<br>260 | 0.0124 | 0.0211 | 16.5 cps | 0.0207<br>0.007 | 0.0353<br>0.0119 | 16.6 cps<br>123 cps |
| On Hopper Side (Transit in Channel) | 94 | 0.0198* | - | 38 cps | 0.0055* | - | 58 cps |
| In Probe (Transit in Channel) | 95 | 0.0124 Beats<br>0.0031 | 0.0211<br>0.0053 | 8th (12.6 cps) | 0.0083 | 0.0141 | 8th(12.6 cps) |
| On Hopper Side (Transit to S.F.) | 188 | 0.0494* | - | 8th (25 cps)<br>12th (37 cps) | 0.0070* | - | 8th(25 cps)<br>12th(37 cps) |
| In Probe (Tran. to S.F.) | 189 | 0.0109 | 0.0186 | Mixed | 0.0242 | 0.0412 | 8th(25 cps) |
| In Probe (Tran. to S.F.) | 187 | 0.0080 | 0.0136 | 4th (12.5 cps)<br>8th (25 cps)<br>54 cps | 0.030 | 0.0511 | 8th(25 cps) |
| On Hopper Side (Transit to S.F.) | 187 | 0.0565* | - | 8th (25 cps) | 0.0135* | - | 8th(25 cps) |
| Probe | 94 | 1.200 | - | 0.7 cps (Reson) | 0.0794 | - | 0.7cps(Reson) |
| Dumping | | 0.094 | - | 7 cps | 0.0276 | - | 145 cps |
| Early Part of Dump | → | 0.124 | 0.211 | 12.6 cps | 0.0310 | 0.0528 | High |

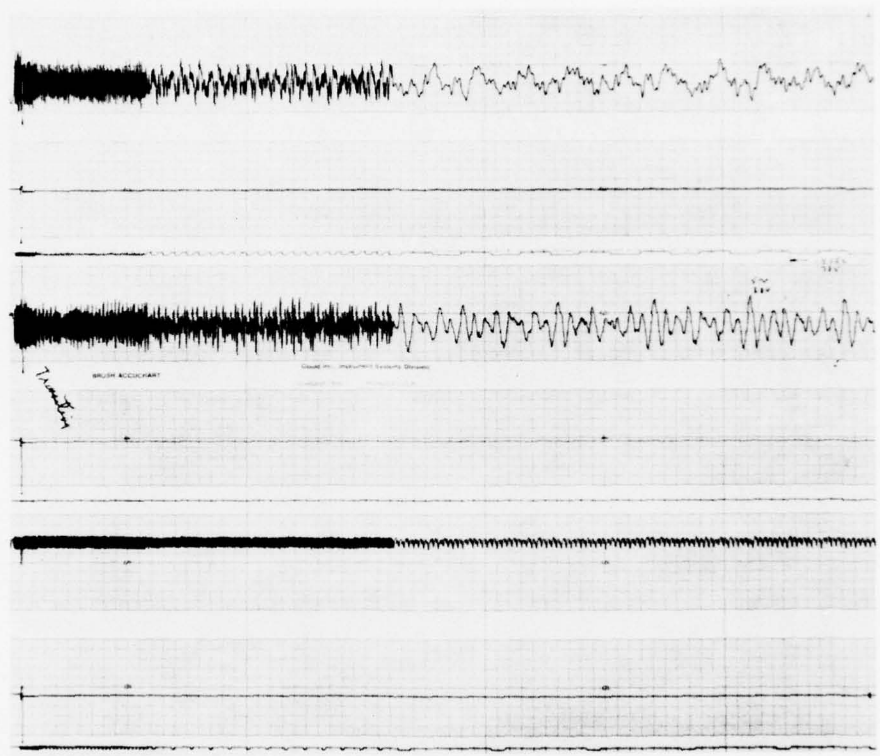


Figure 3-39. Vibration Record from HARDING

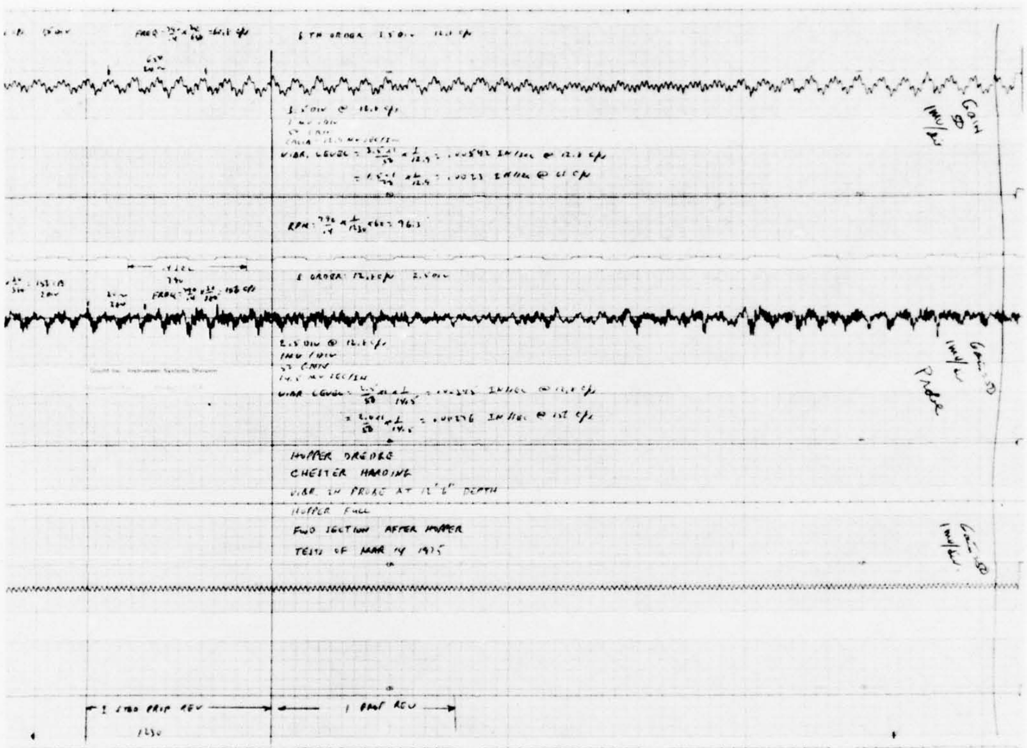
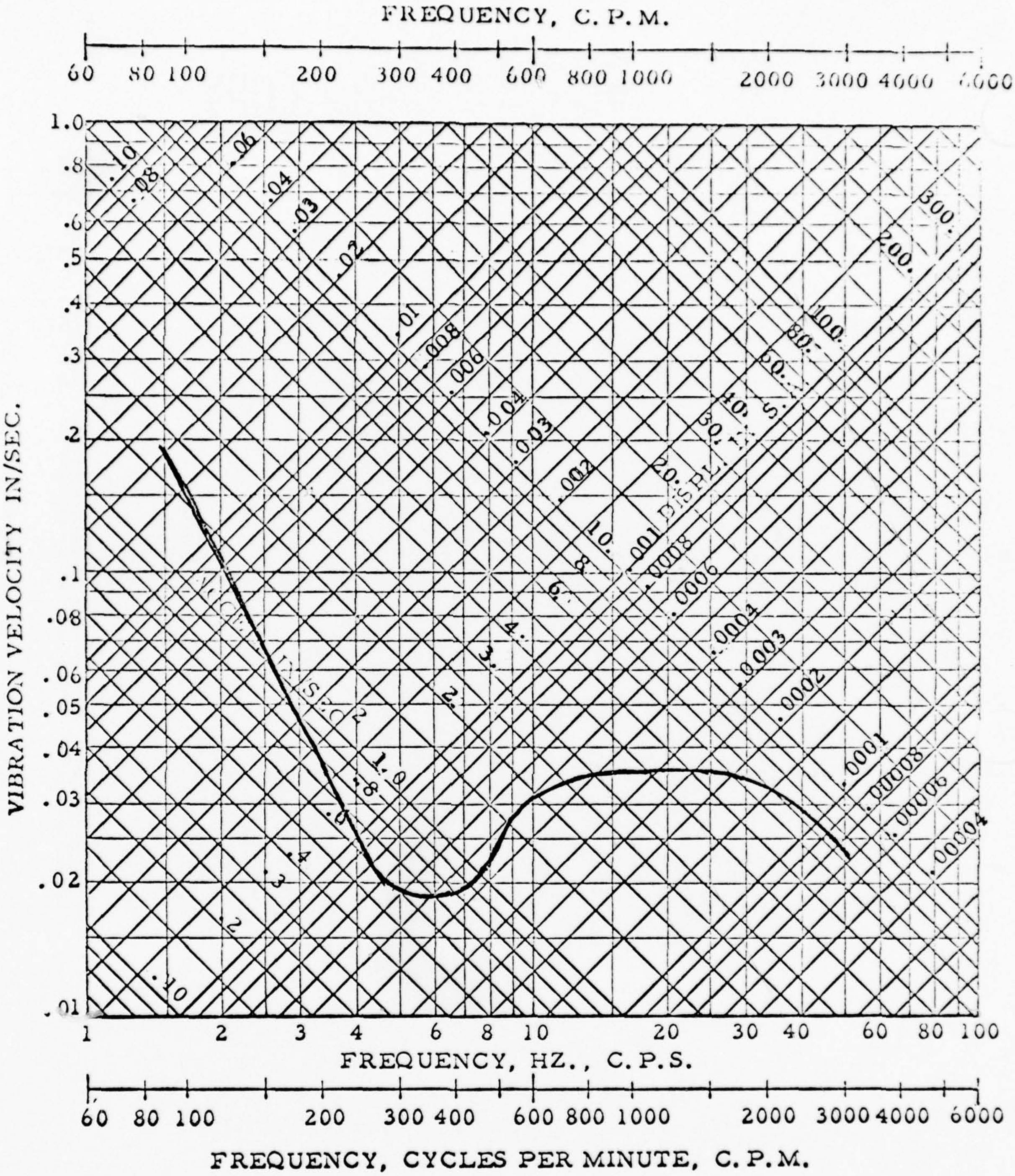



Figure 3-40. Vibrations on the HARDING




VIBRATION PERCEPTION LEVEL

(Source)
The Effects of Shock and Vibration on Man
Goldman, D. E. and vonGierke, H. E. (1960)
United States of America Standards Institute

Figure 3-41

It is apparent from Table 3-13 that the frequencies and amplitudes measured varied extensively during the test period. Some of the highest vibrations were experienced during the period that the pumps were running (dredging) and during maneuvers while transiting.

Frequencies observed ranged from slightly under one hertz to 158 hertz but those which appeared to be most prevalent were in the 15 to 30 hertz range. In all cases, however, the amplitudes were low.

The velocities from Table 3-13 can be converted to displacement values by the following relationship:

$$\text{displacement} = \frac{\text{velocity}}{2\pi f}$$

where f is the frequency in hertz

Table 3-14 shows the displacement amplitudes calculated from Table 3-13 for the case of the hoppers full and measurements being made on the walls and in the dredged material.

The levels in the sediment are lower than on the walls, indicating that there is a coupling loss and probably attenuation of the vibrations in the sediment. The fore and aft displacement in the sediment at 25 feet, for the data shown, does not follow this pattern but, in general, the levels in the dredged material are lower. For any of the data in Table 3-13, the vibrations are of extremely low amplitude.

### 3. Laboratory Simulation of Vessel Vibrations

#### General

A detailed laboratory simulation program was conducted to investigate

## TABLE 3-14

### Hopper Dredge Vibration Displacement Amplitudes

(in inches x $10^{-6}$)

| Depth In Hopper | Transverse | | Fore and Aft | |
|---|---|---|---|---|
| | Wall | Sediment | Wall | Sediment |
| 12.5 | 190 | 110 | 87 | 70 |
| 25 | 150 | 76 | 37 | 86 |

the effects, if any, of vibration on the dredged material in barges and hopper dredges while transiting to the dump site. Frequencies and amplitudes were selected based upon measurements made on the HARDING. Three frequencies and amplitudes were selected:

$\pm$ 2.5 inches at 0.2 hertz
$\pm$ 0.001 inches at 25 hertz
$\pm$ 0.001 inches at 60 hertz

If an effect were to be observed, it was felt that this would occur in the low frequency region, representing vessel motions in a seaway. For this reason, the 0.2 hertz frequency was selected and the test run at the maximum displacement that the MTS machine could provide. As shown in Table 3-14, displacement amplitudes experienced on the HARDING were on the order of 0.00001 inches. The level selected for 25 and 60 hertz was an order of magnitude larger than observed on the HARDING.

The laboratory program consisted of three parts:

- sediment index and classification
- thixotropic effects
- sample vibrations

These are discussed in the following sections.

## Sediment Index and Classification

A complete index and classification test program was carried out on samples from all bulk barrel samples. The tests consisted of water content and dry density determinations, Atterberg Limits and grain size determinations. Figure 3-42 presents the results of the Liquid Limit and Plastic Limit tests which are referred to as Atterberg Limits. The tests are carried out in accordance with ASTM D-423-66 (Reapproved 1972) "Standard Method of Test for Liquid Limit of Soils" and ASTM D-424-59 (Reapproved 1971)

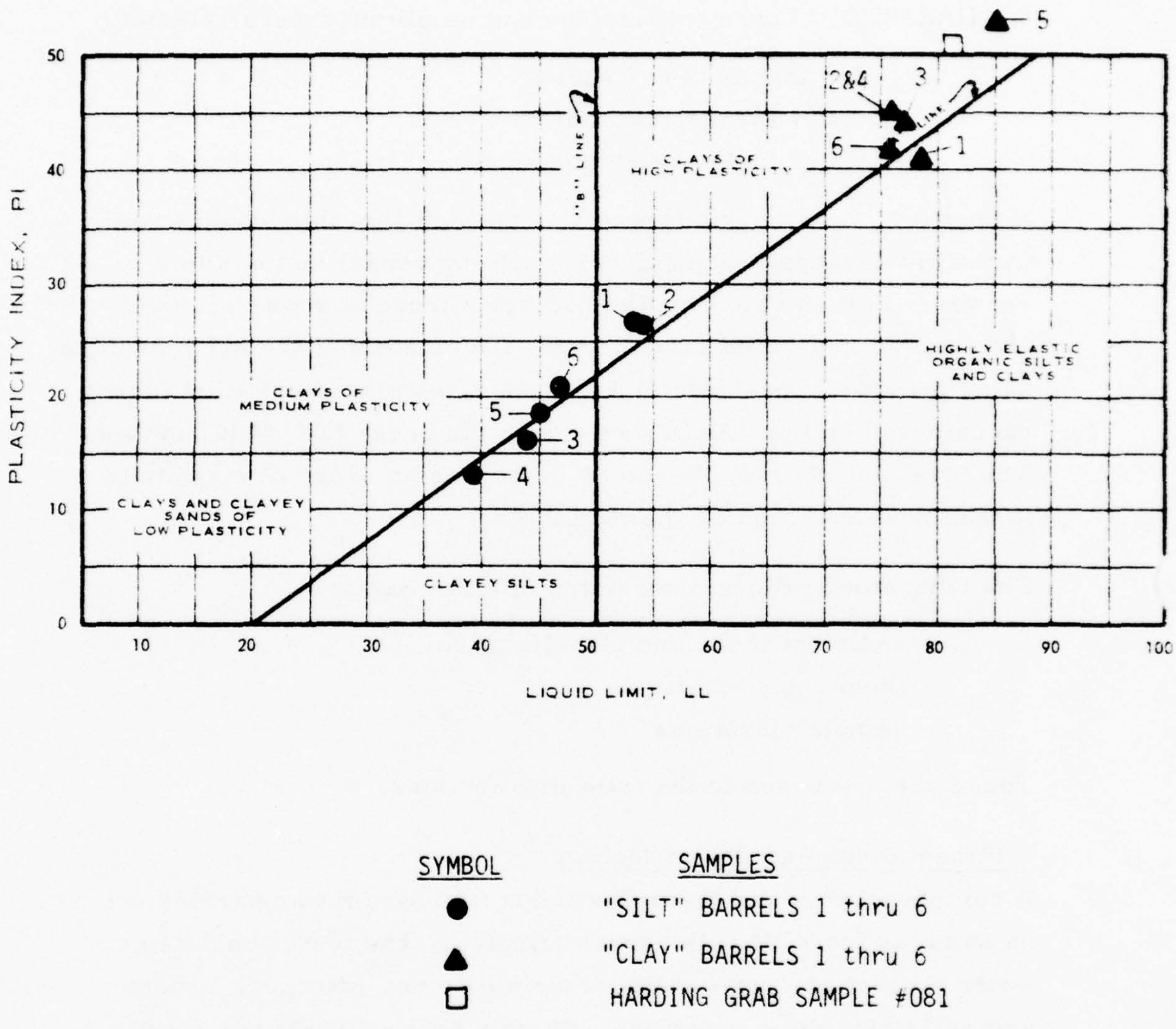



Figure 3-42. Atterberg Limits (Plasticity Index = Liquid Limit - Plastic Limit).

"Standard Method of Test for Plastic Limit and Plasticity Index of Soils". The results of the Atterberg Limit tests are plotted as Plasticity Index vs Liquid Limit (Figure 3-42) in order to determine the soil classification in accordance with the Unified Soil Classification System (refer to ASTM D-2487-69 for a description of the Unified Classification System). All barrel samples tested were classified as silty or highly plastic clays.

Figures 3-43 through 3-46 present results of standard hydrometer analyses on barrel samples. Information from the grain size analyses is also used to help classify soils in accordance with the Unifified Classification System. Table 3-15 presents all index and classification data on the barrel samples, including the Unified Soil Classification for each sample. The note in Table 3-15 regarding time of sample storage before testing is meant to direct attention to the fact that the considerable length of storage without disturbance may have had some effect on the water content and dry density of the materials before they were tested, due to sedimentation and possibly some consolidation. It may also have had a considerable effect on in situ shear strength, as this material is known to be highly susceptible to thixotrophy (ASTM D-653).

Figures 3-47 through 3-50 present the results of laboratory vane shear tests performed at three depths in each barrel after about four weeks of undisturbed storage. The shear strength values obtained were generally higher than those recorded in the field. They were distinctly measurable even though the in situ water content of all barrel samples was well in excess of the liquid limit of the respective materials; in fact, it was closer to twice the liquid limit in most cases. Since the soils at a water content above the liquid limit are expected to behave as a fluid with zero shear strength, only a thixotropic strength gain during the four week

| SAMPLE NO. | SYMBOL | DEPTH | LL | PI | UNIFIED CLASSIFICATION |
|---|---|---|---|---|---|
| SILT BARREL #1 | ———— | ——— | 53 | 27 | CH |
| SILT BARREL #2 | — — — — | ——— | 54 | 27 | CH |
| SILT BARREL #3 | - - - - - - | ——— | 44 | 16 | ML |

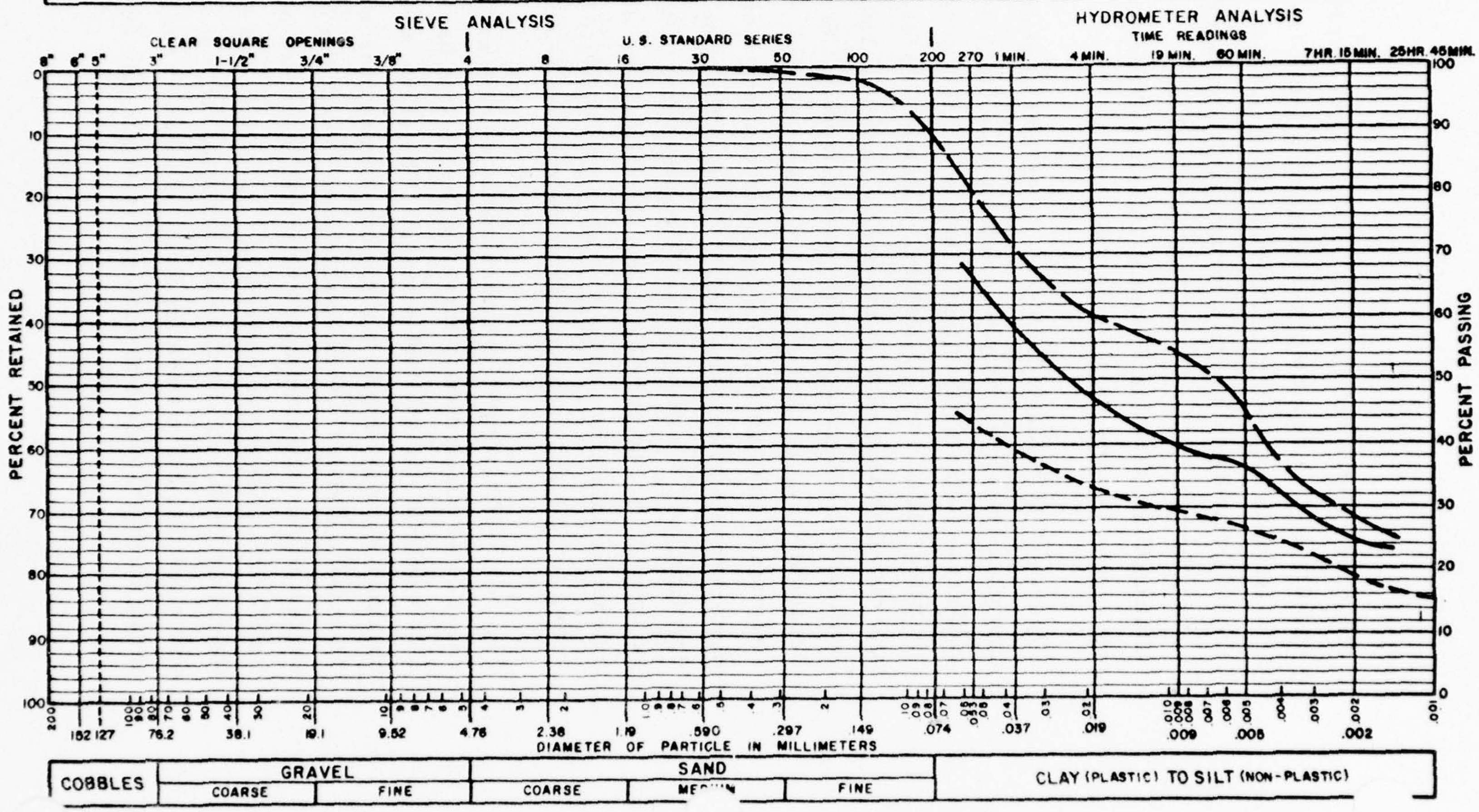



Figure 3-43. Grain Size Anal, .s - "Silt" Barrel Samples

| SAMPLE NO. | SYMBOL | DEPTH | LL | PI | UNIFIED CLASSIFICATION |
|---|---|---|---|---|---|
| SILT BARREL #4 | ——— | —— | 39 | 13 | ML |
| SILT BARREL #5 | — — — | —— | 45 | 18 | CL |
| SILT BARREL #6 | - - - - - - | —— | 47 | 21 | CL |

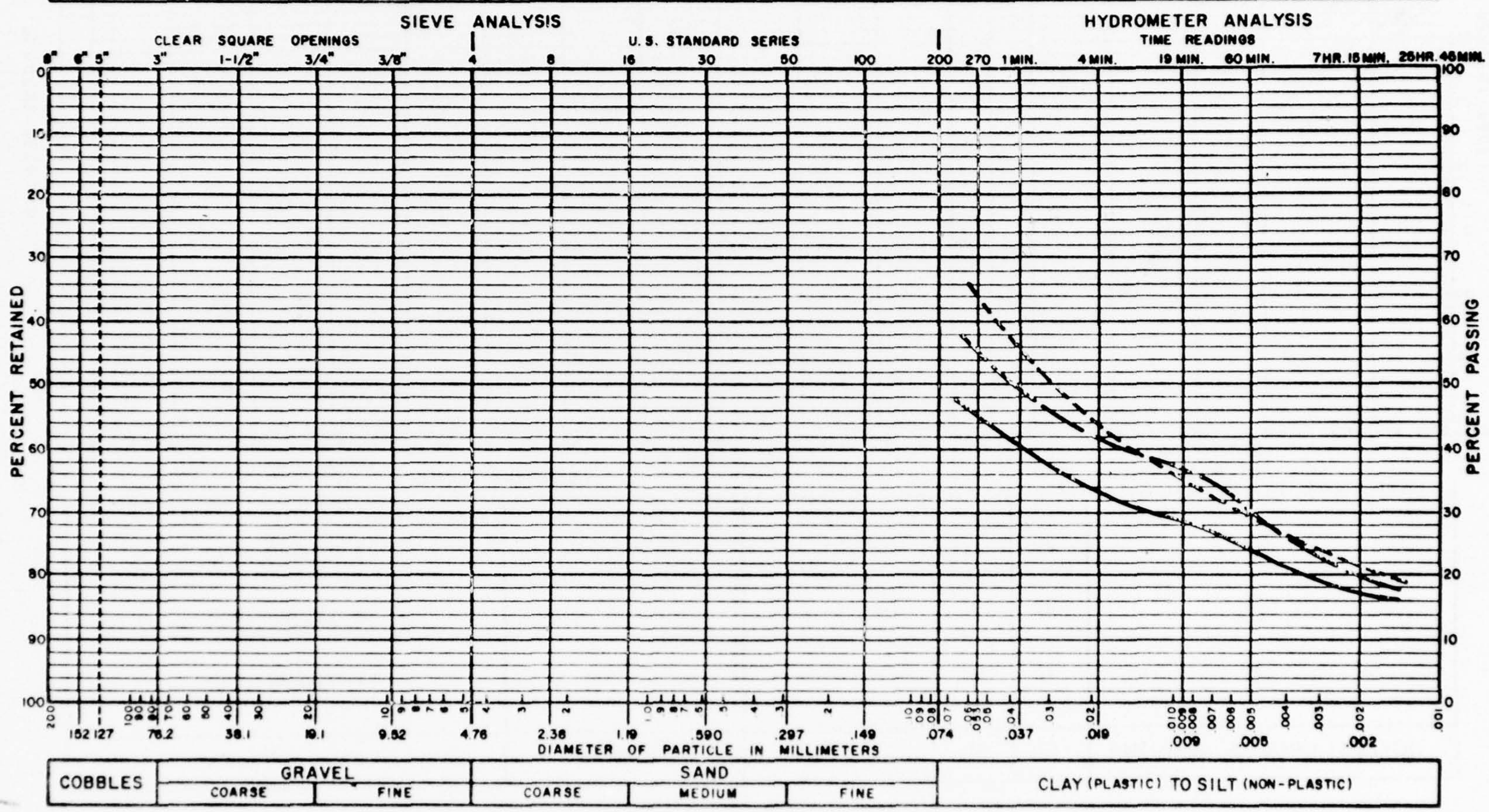



Figure 3-44. Grain Size Analyses - "Silt" Barrel Samples

| SAMPLE NO. | SYMBOL | DEPTH | LL | PI | UNIFIED CLASSIFICATION |
|---|---|---|---|---|---|
| CLAY BARREL #1 | ——— | — | 78 | 41 | MH |
| CLAY BARREL #2 | — — — | — | 76 | 45 | CH |
| CLAY BARREL #3 | - - - - | — | 77 | 44 | CH |

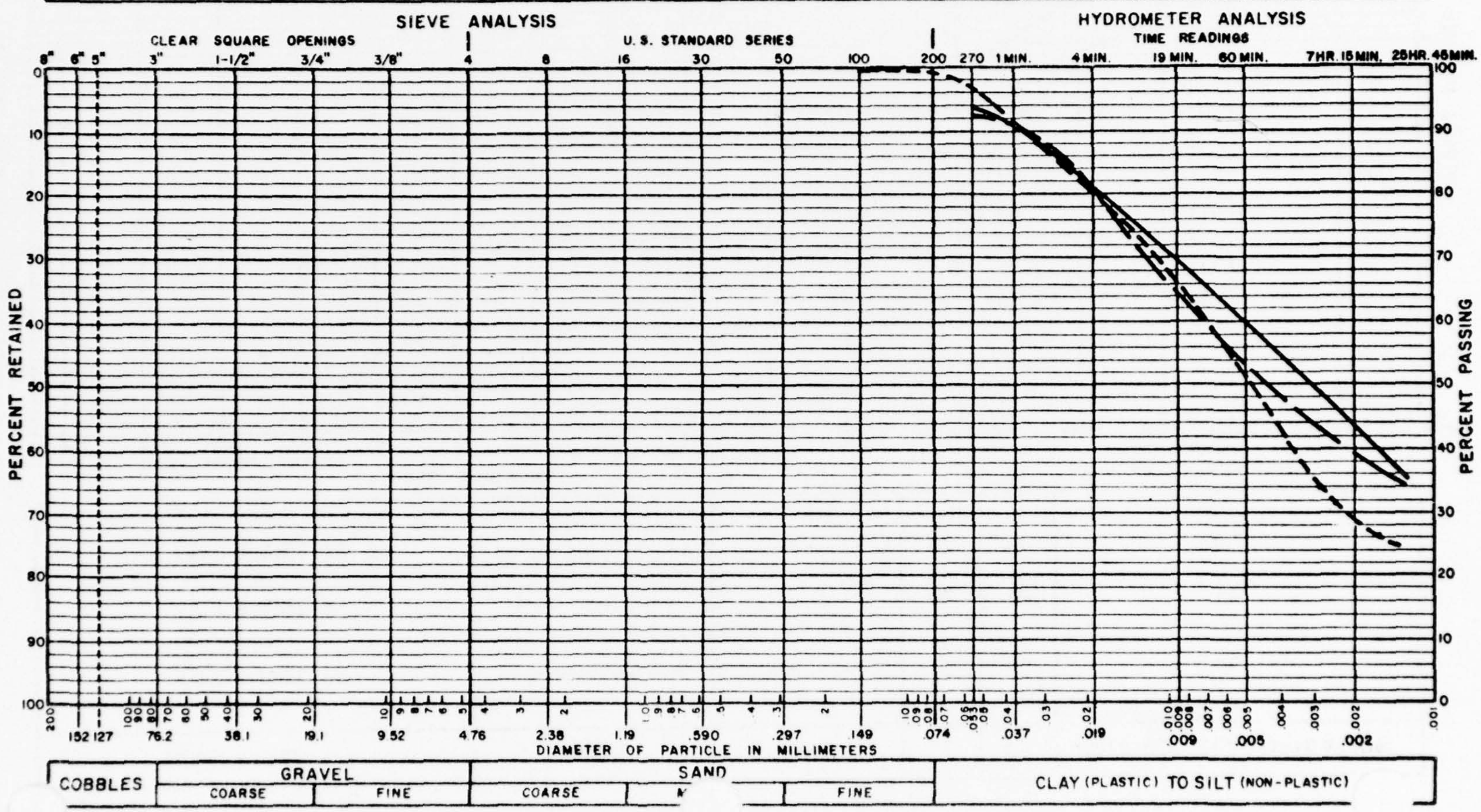



Figure 3-45. Grain Size Anai, ses - "Clay" Barrel Samples

| SAMPLE NO. | SYMBOL | DEPTH | LL | PI | UNIFIED CLASSIFICATION |
|---|---|---|---|---|---|
| CLAY BARREL #4 | ——— | — | 76 | 45 | CH |
| CLAY BARREL #5 | — — — | — | 85 | 53 | CH |
| CLAY BARREL #6 | - - - - | — | 76 | 42 | CH |

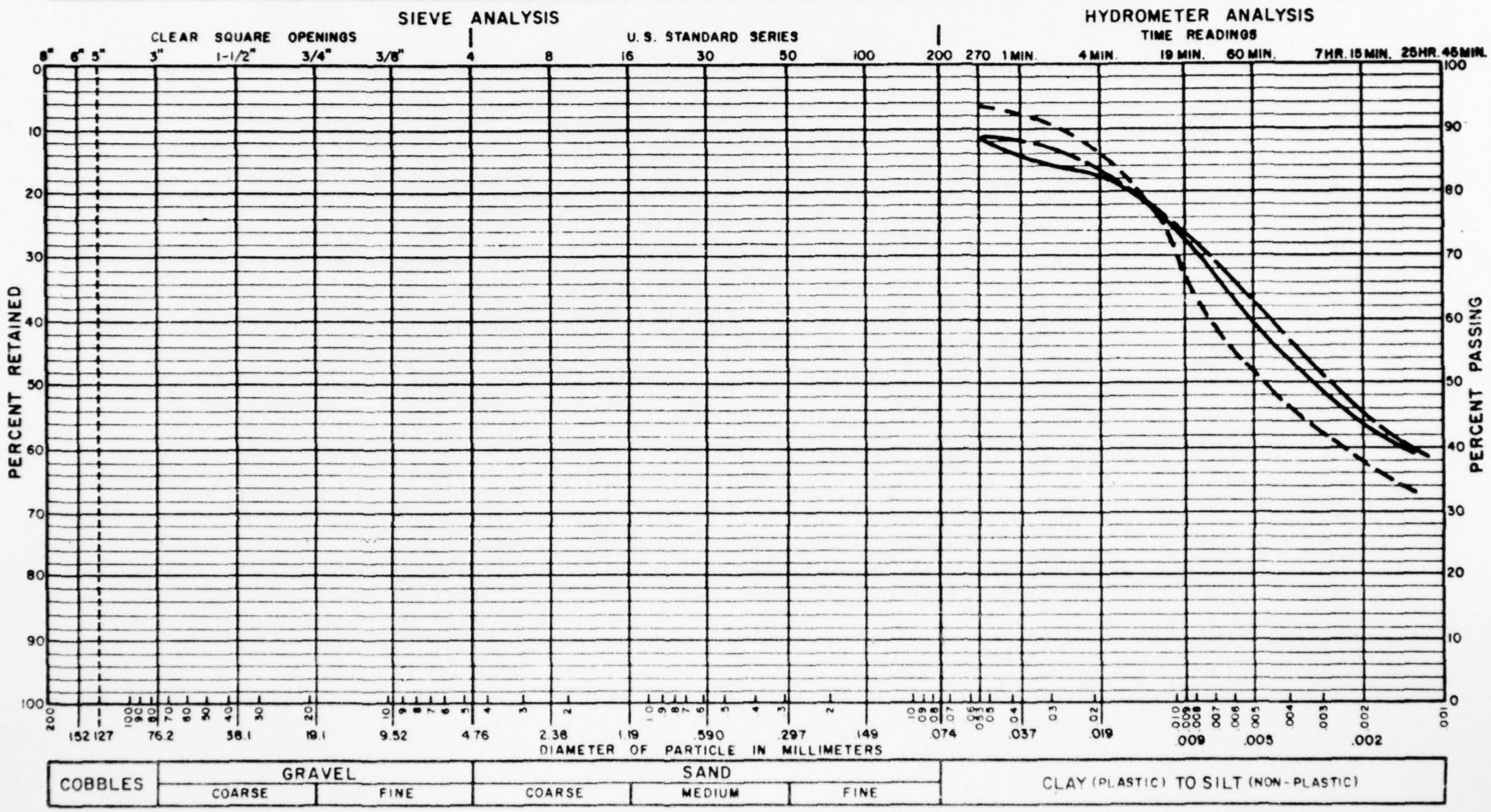



Figure 3-46. Grain Size Analyses - "Clay" Barrel Samples

TABLE 3-15

Physical Properties and Classification of Barrel Samples

| BARREL | LL % | PI % | CLASSIF. | % > 200 | CLAY % SIZES | WC % | DD (PCF) |
|---|---|---|---|---|---|---|---|
| Silt 1 | 53 | 27 | CH | 69 | 36 | 77 | 55 |
| Silt 2 | 54 | 27 | CH | 89 | 45 | 101 | 45 |
| Silt 3 | 44 | 16 | ML | 56 | 27 | 83 | 51 |
| Silt 4 | 39 | 13 | ML | 57 | 24 | 111 | 42 |
| Silt 5 | 45 | 18 | CL | 66 | 30 | 84 | 52 |
| Silt 6 | 47 | 21 | CL | 81 | 30 | 76 | 54 |
| Clay 1 | 78 | 41 | MH | 98 | 60 | 170 | 36 |
| Clay 2 | 76 | 45 | CH | 99 | 54 | 123 | 39 |
| Clay 3 | 77 | 44 | CH | 99 | 51 | 122 | 39 |
| Clay 4 | 76 | 45 | CH | 98 | 60 | 129 | 37 |
| Clay 5 | 85 | 53 | CH | 99 | 62 | 135 | 36 |
| Clay 6 | 76 | 42 | CH | 99 | 52 | 127 | 38 |

NOTE: All barrels had been stored for a period of approximately 4 weeks before being transported to WCC. They were then stored for an additional 4 weeks before any testing on the silts and clays was done.

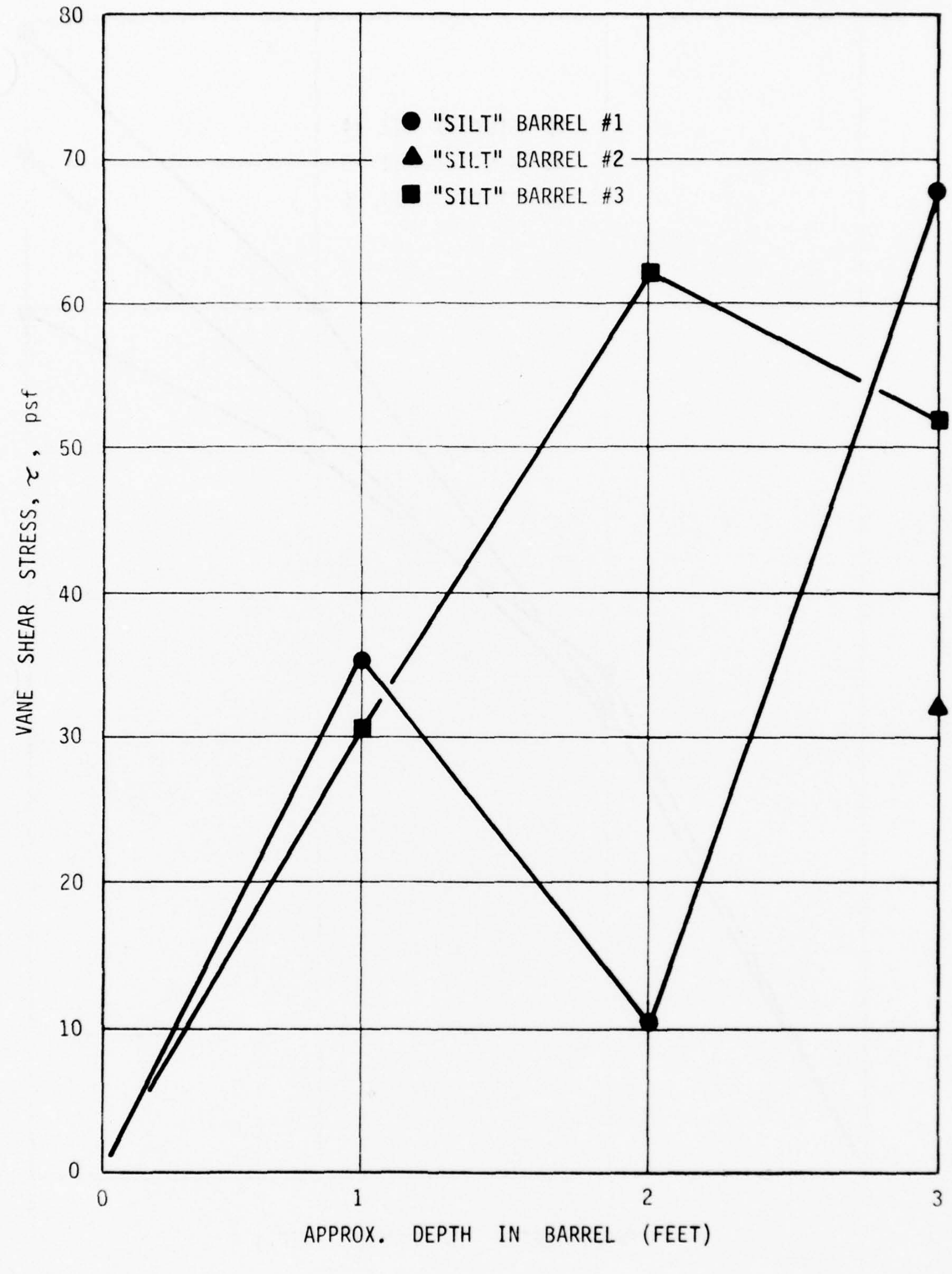



Figure 3-47. Laboratory Vane Shear Tests - "Silt" Barrels

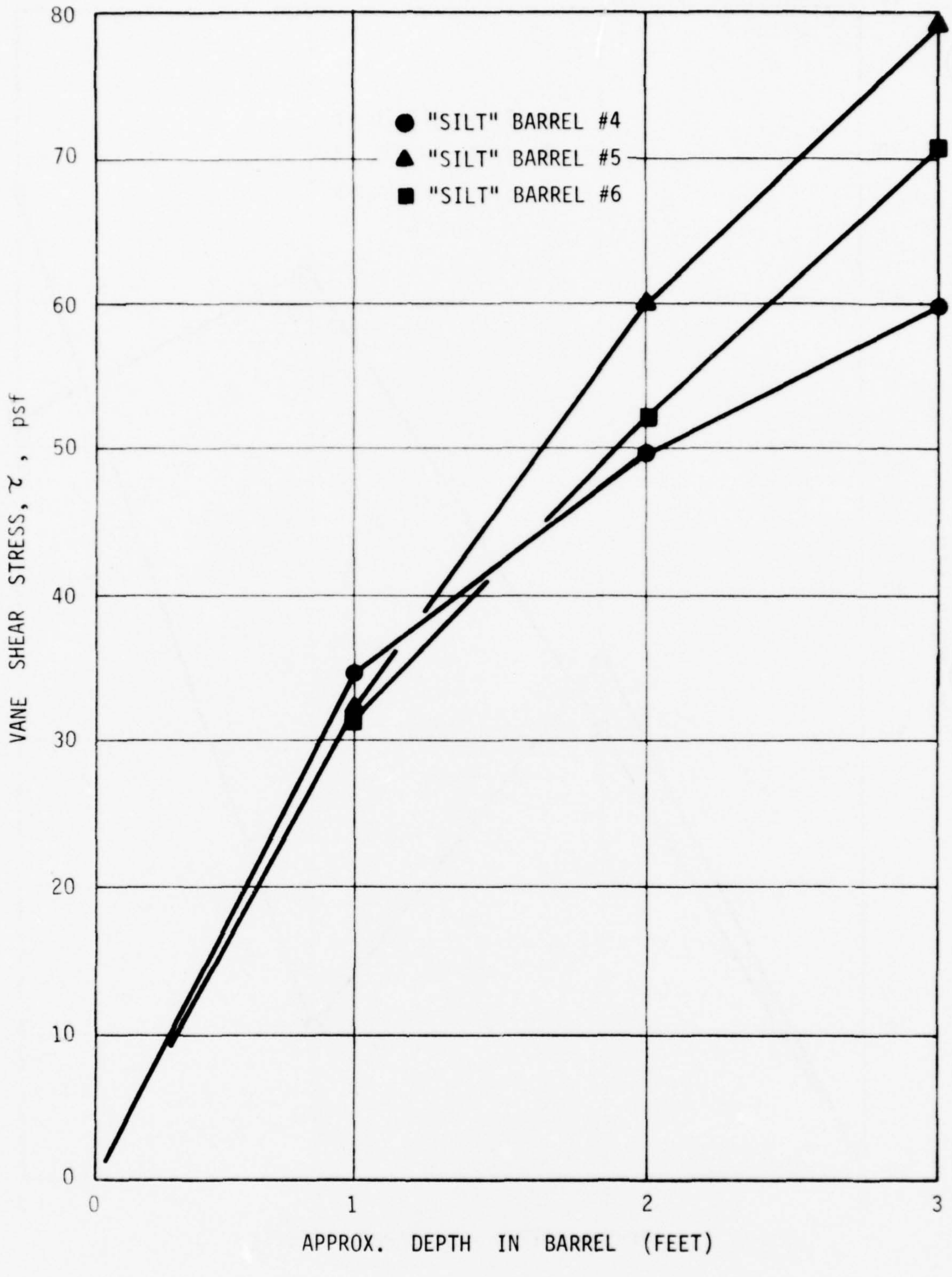



Figure 3-48. Laboratory Vane Shear Tests - "Silt" Barrels

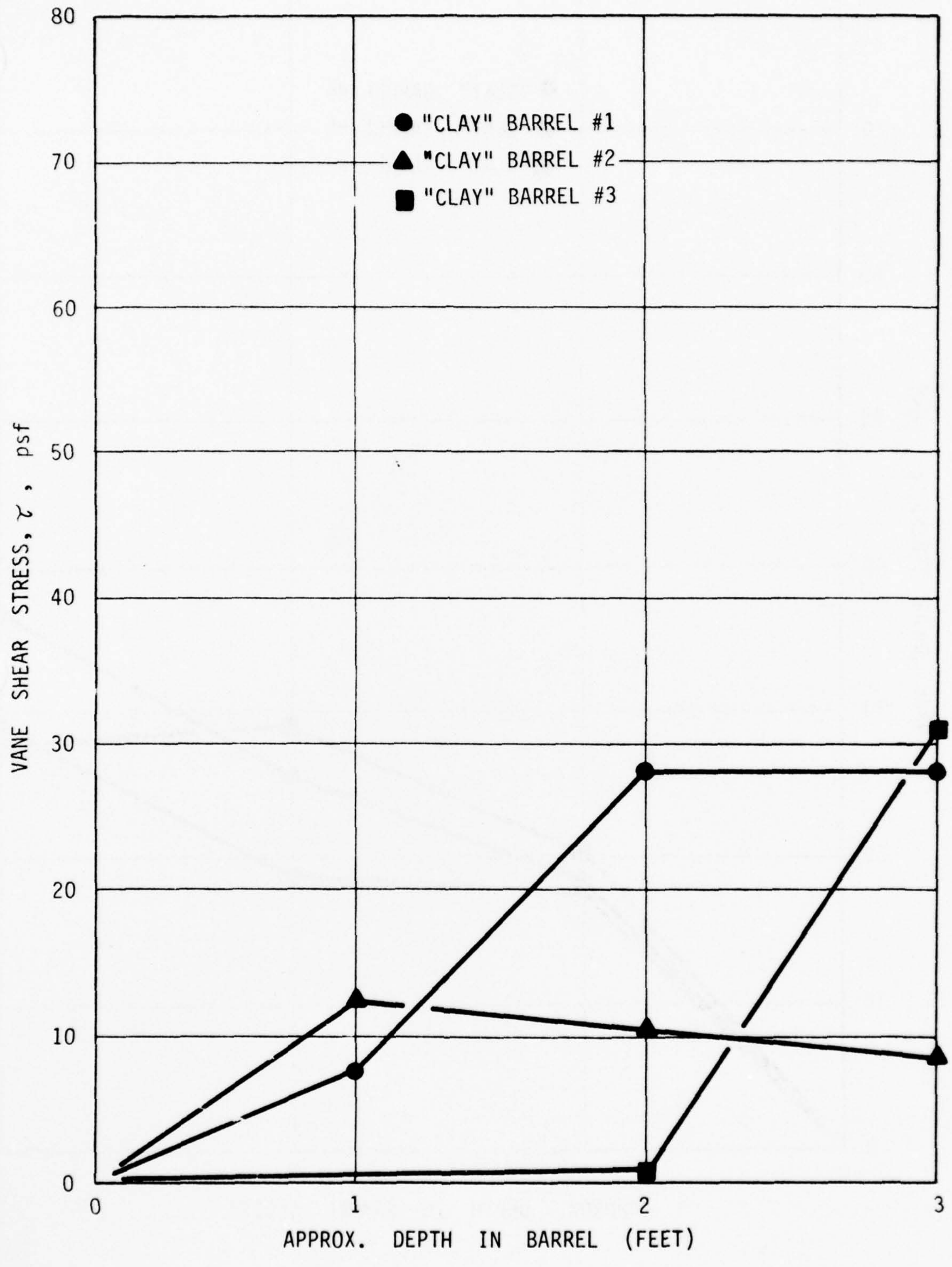



Figure 3-49. Laboratory Vane Shear Tests - "Clay" Barrels

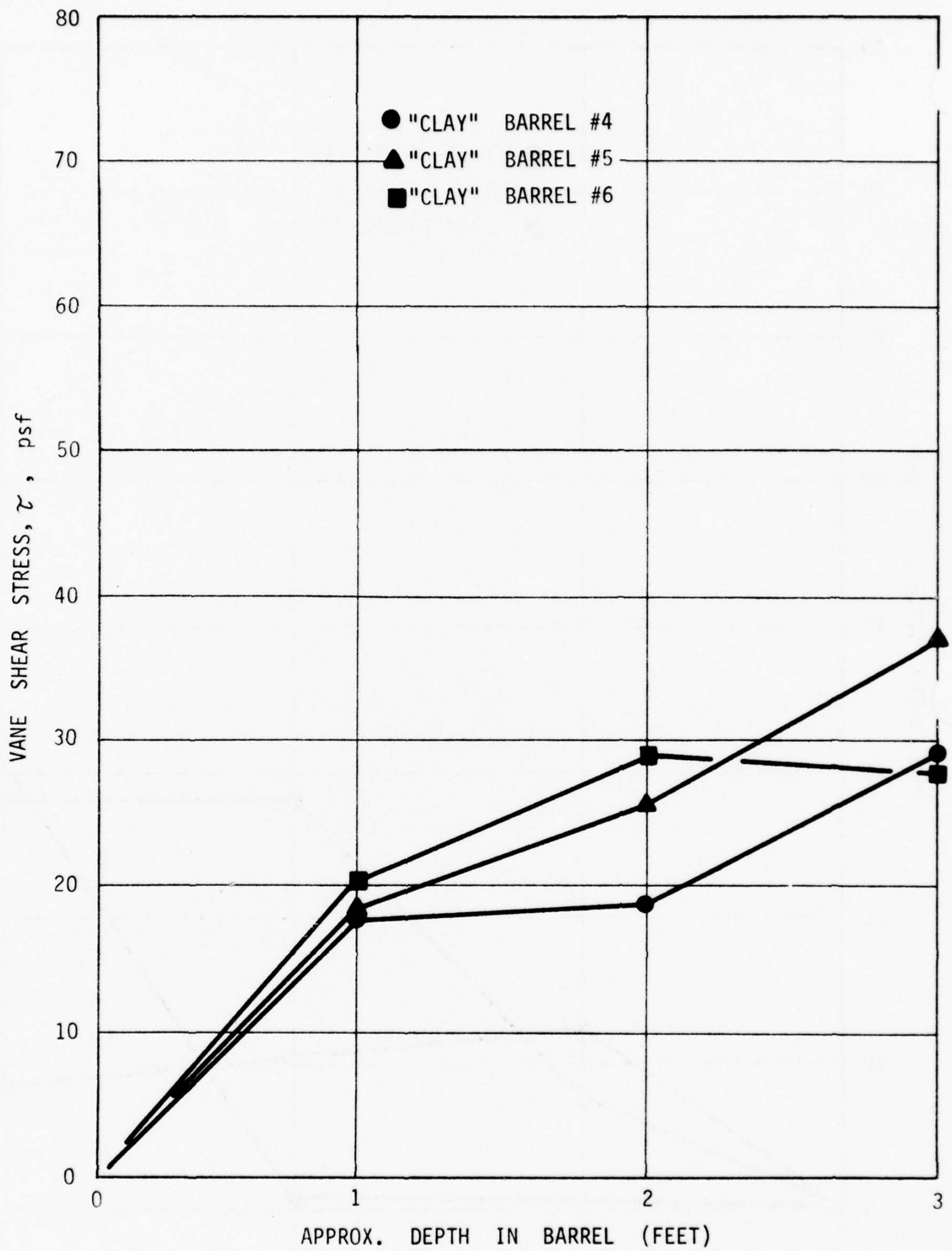



Figure 3-50. Laboratory Vane Shear Tests - "Clay" Barrels

storage can account for the results obtained. However, the effects of such a strength gain can be readily eliminated with any sort of agitation.

A summary of water content and dry density data is presented in Figure 3-51. All the measured water contents of the 50 cc syringe samples from the dredges, the barrel samples, and the diver samples were plotted against the respective calculated dry densities. The data were found to be consistent enough to justify drawing average curves, which could then be used to find an approximate dry density for a given water content or vice versa. The curve marked "Average for Barrel Samples" was in fact used to obtain the dry densities reported in Appendix A for the "BU" core samples for which no sample volume had been available.

## Thixotropic Effects

Since the materials involved in this study are known to be highly susceptible to thixotropy, laboratory shear vane tests were performed on the silts and clays in order to determine whether this strength gain could occur within the time period of a 60-mile haul distance, at 9 knots. Figures 3-52 through 3-55 present the results of these tests. Shear strength was plotted against elapsed time between tests and the series was carried to 24 hours which greatly exceeds the time of the simulated haul of 7 hours. No strength gain was noted in either silt or clay samples, indicating that any thixotropic effect was negligible within a 24-hour time period. The only difference noted was the uniformity of the results of the clay slurry (Tests A and B) which simulated the hopper dredge conditions, as opposed to the more scattered results of the clay and silt dumped at random (Tests D and C) which simulated the clamshell dredge operation.

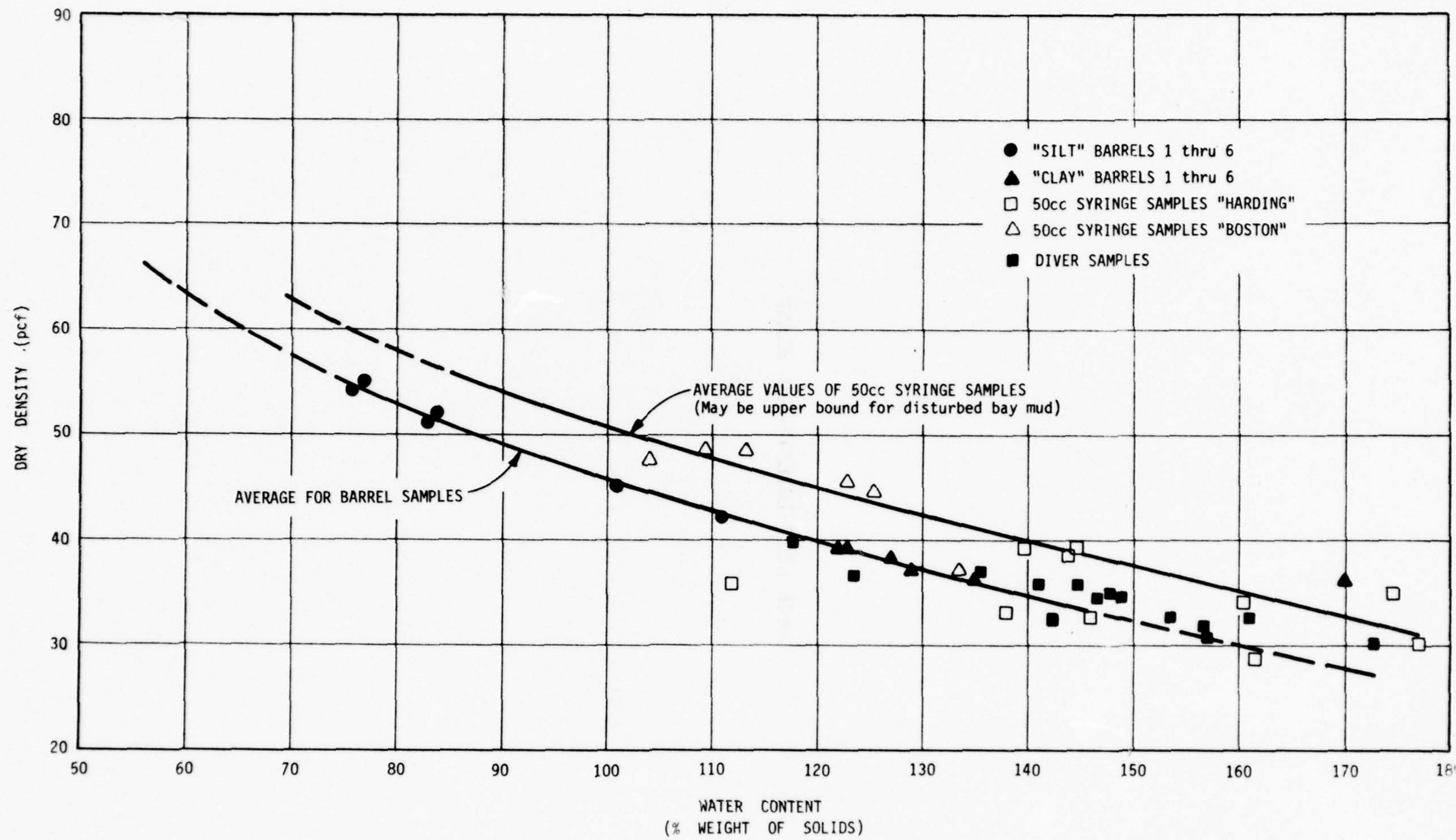



Figure 3-51. PHYSICAL CHARACTERISTICS - IN SITU DREDGE SAMPLES, BARREL SAMPLES, AND DIVER SAMPLES

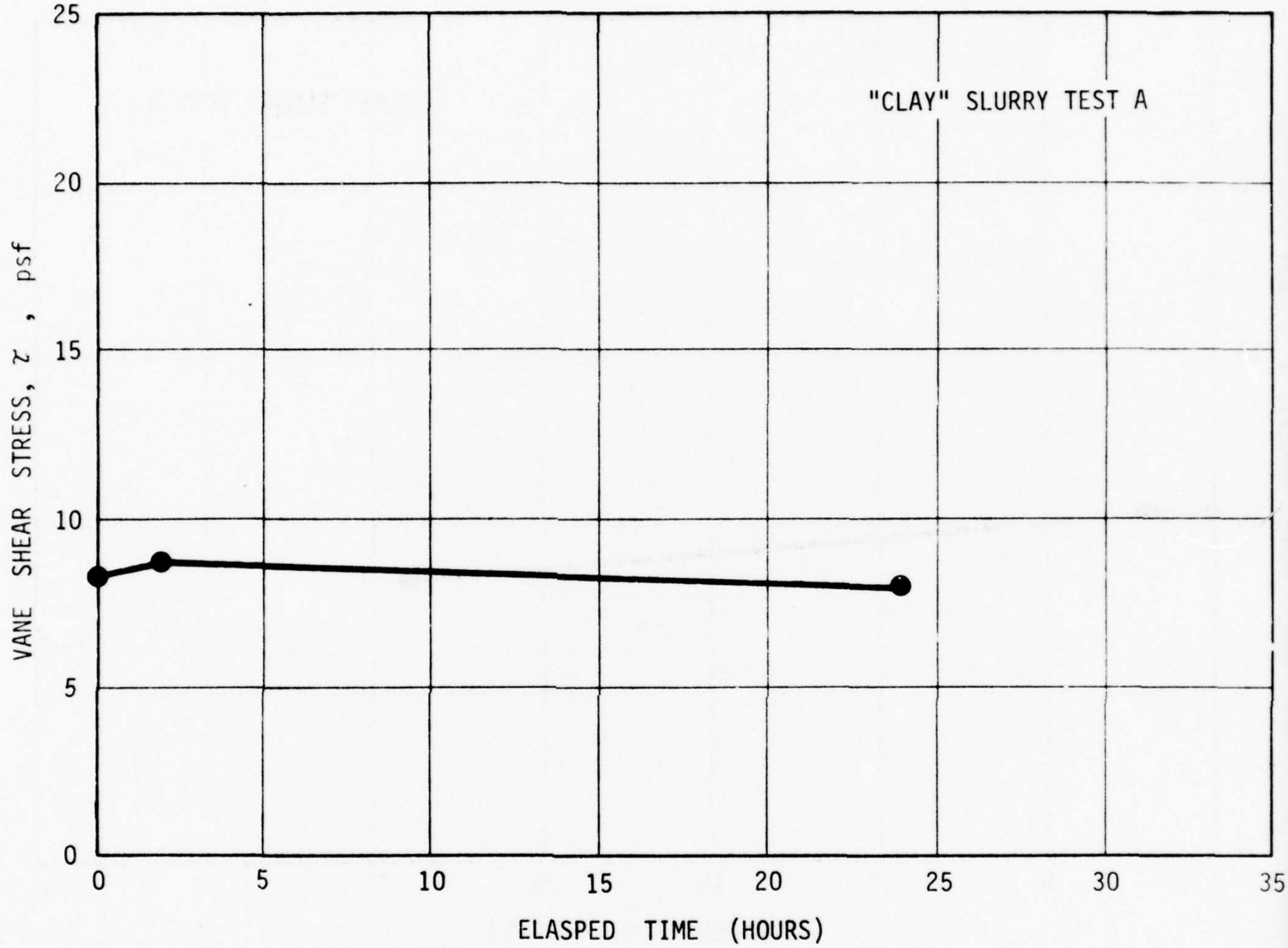



Figure 3-52. LABORATORY VANE SHEAR TESTS FOR THIXOTROPIC EFFECTS

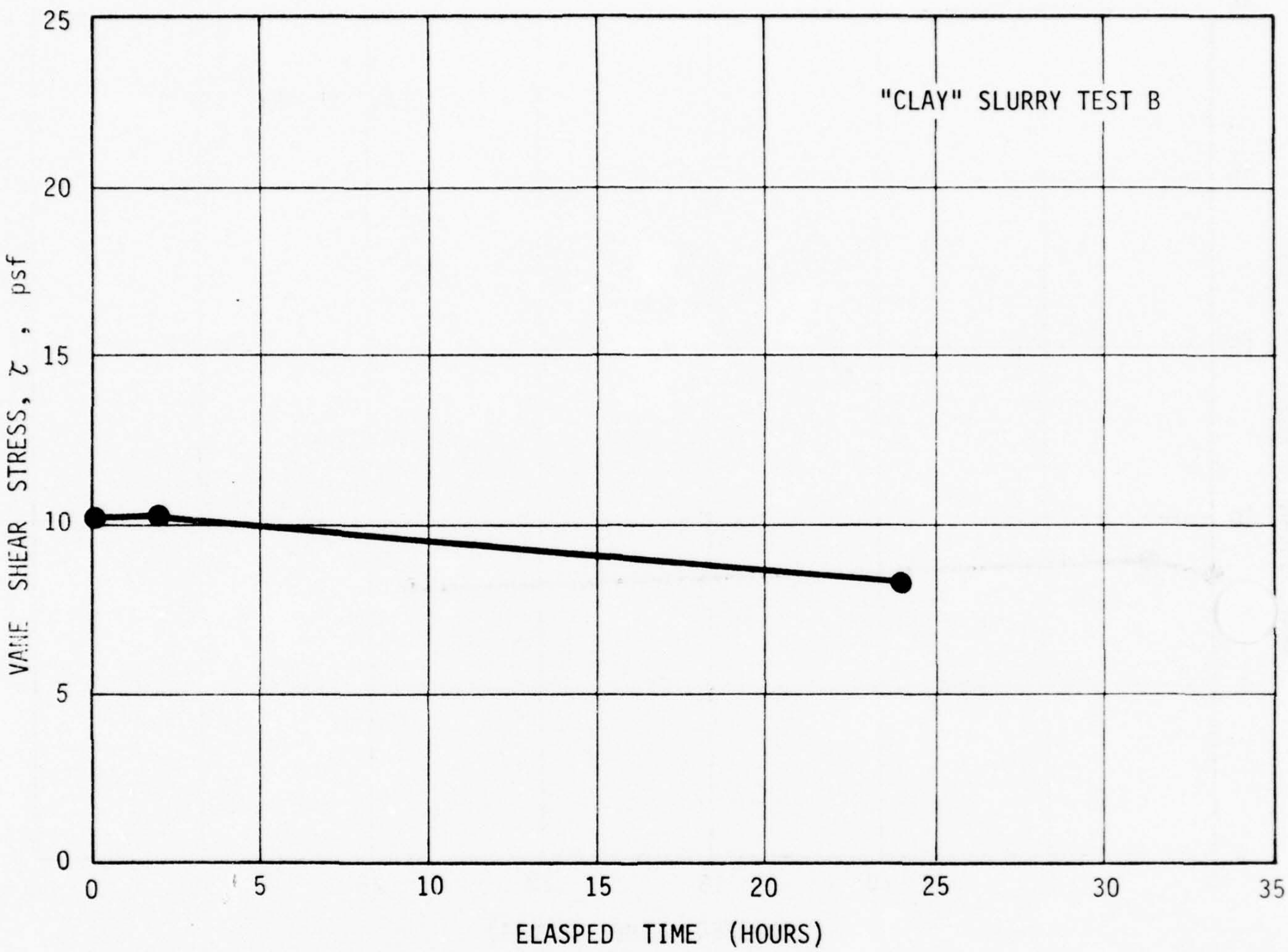
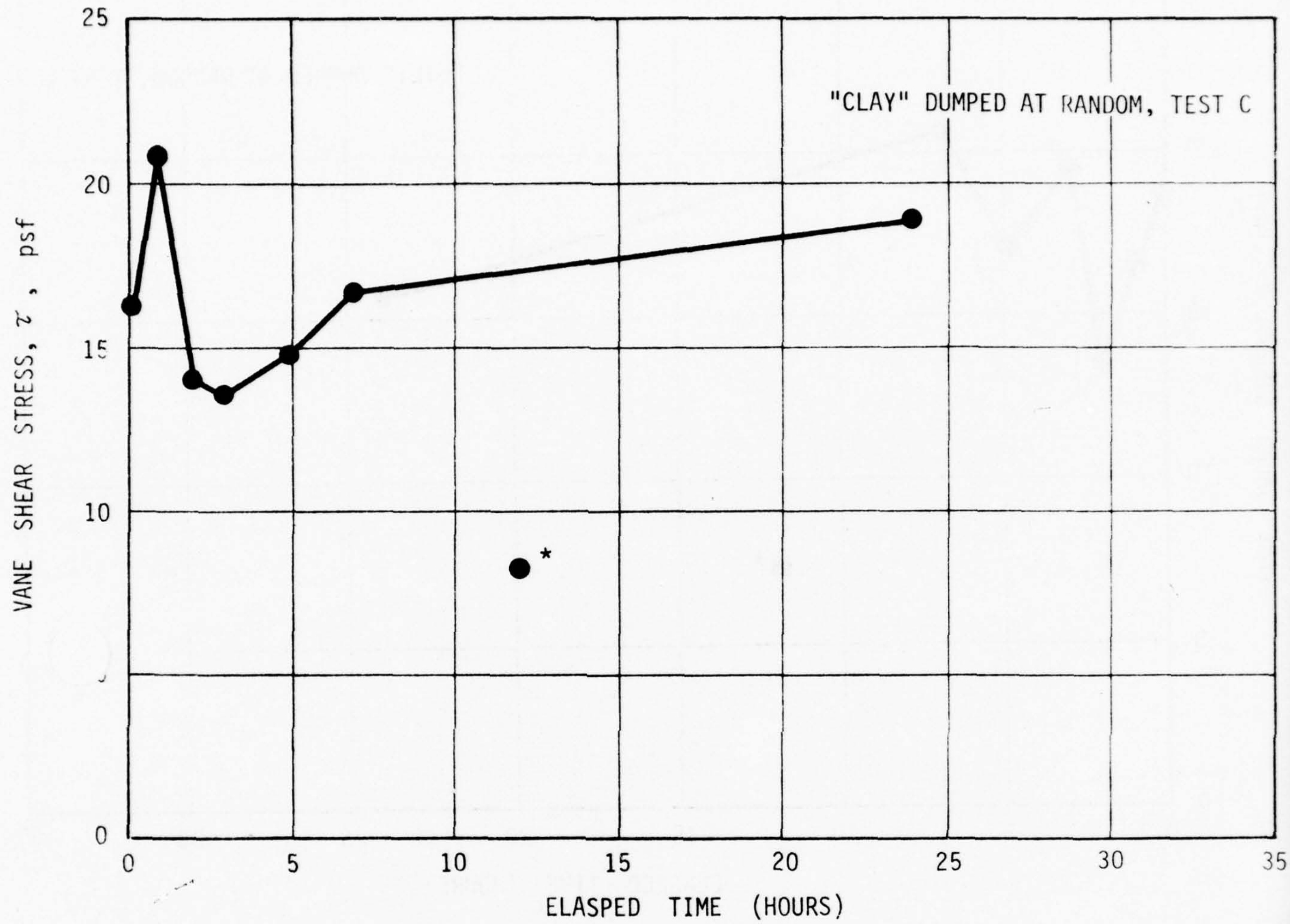
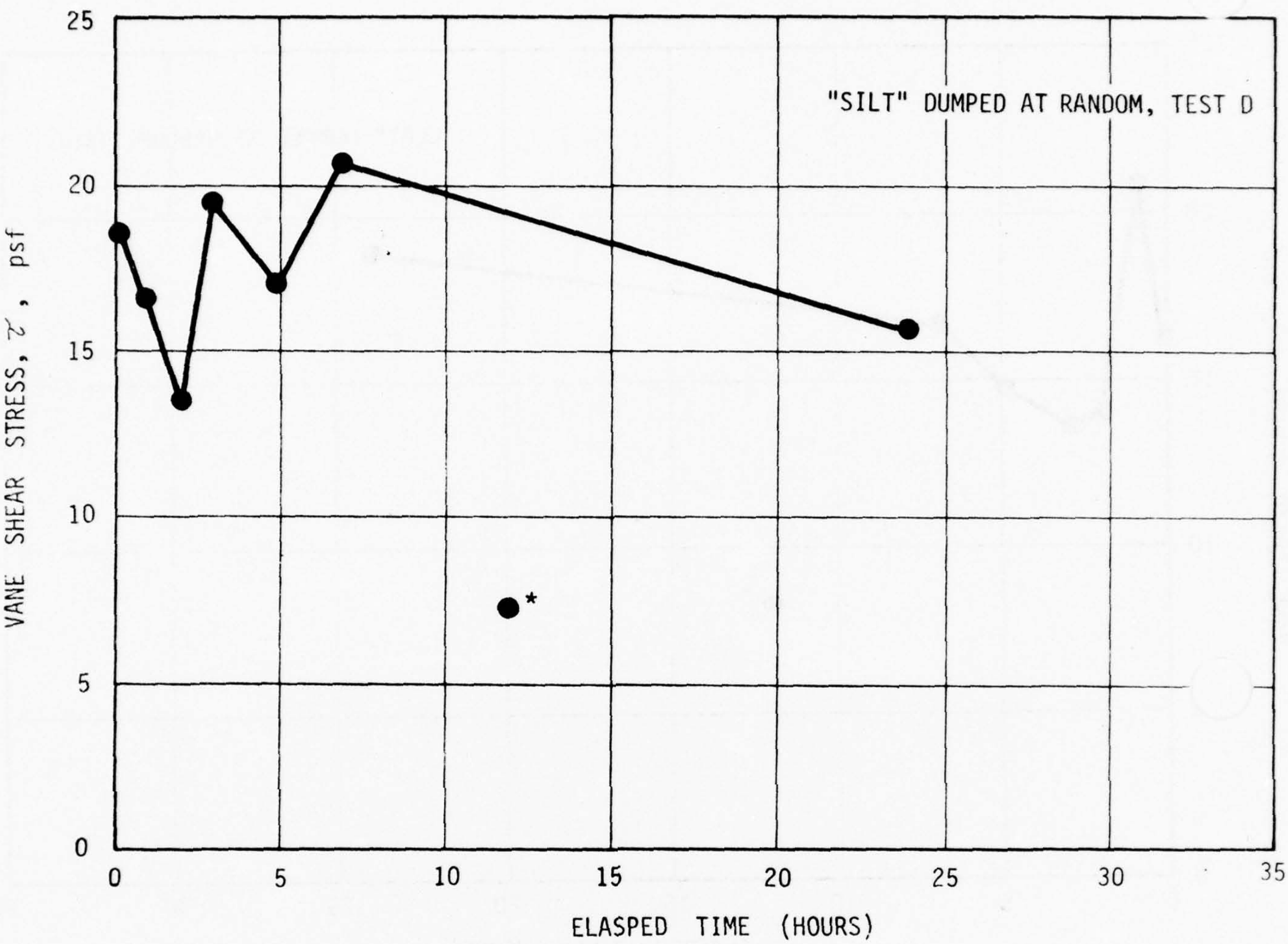



Figure 3-53. LABORATORY VANE SHEAR TESTS FOR THIXOTROPIC EFFECTS




*READING BY DIFFERENT OPERATOR

Figure 3-54. LABORATORY VANE SHEAR TESTS FOR THIXOTROPIC EFFECTS




*READING BY DIFFERENT OPERATOR

Figure 3-55. LABORATORY VANE SHEAR TESTS FOR THIXOTROPIC EFFECTS

## Vibrations

The purpose of the MTS program was to simulate in the laboratory the behavior of dredged material during transit from the dredge site to the disposal site. In order to distinguish between time effects and vibration effects, two separate sets of samples were prepared for each test in two-foot high, clear plastic cylinders. One was vibrated for the assigned seven-hour period, while the other was allowed to sit undisturbed for the same time period.

Figure 3-56 shows a typical cylinder with the laboratory shear vane installed on top. It also shows a closeup of the vane shear pulley system. Torque measurements were made by the addition of small weights to a tray suspended from the pulley system.

The water content of each test was standardized to the liquid limit of the materials. One set of tests was run at the liquid limit, which was 47 percent moisture for the silt and 78 percent moisture for the clay. The other sets were run at water contents equal to 2 times, 5 times, and 8 times the liquid limit.

The MTS program is summarized in Table 3-16. A total of 18 tests were run, 9 on silts and 9 on clays. For each test at the given frequency and water content, three samples were recovered from the two-foot high test cylinders (both static and dynamic) before each test (0 hours, Samples A, B, C) and after each test (7 hours, Samples D, E, F). In each case the samples were recovered with a vacuum device from near the top, the middle, and the bottom of each cylinder. The samples were placed in a graduated cylinder to obtain an accurate volume and were weighed to obtain the wet weight. The samples were drawn through a Buechner funnel to collect all suspended solids and then oven dried, and the dry weight recorded. The filter paper used was Whatman No. 1 Paper. The final moisture contents were then calculated based on wet

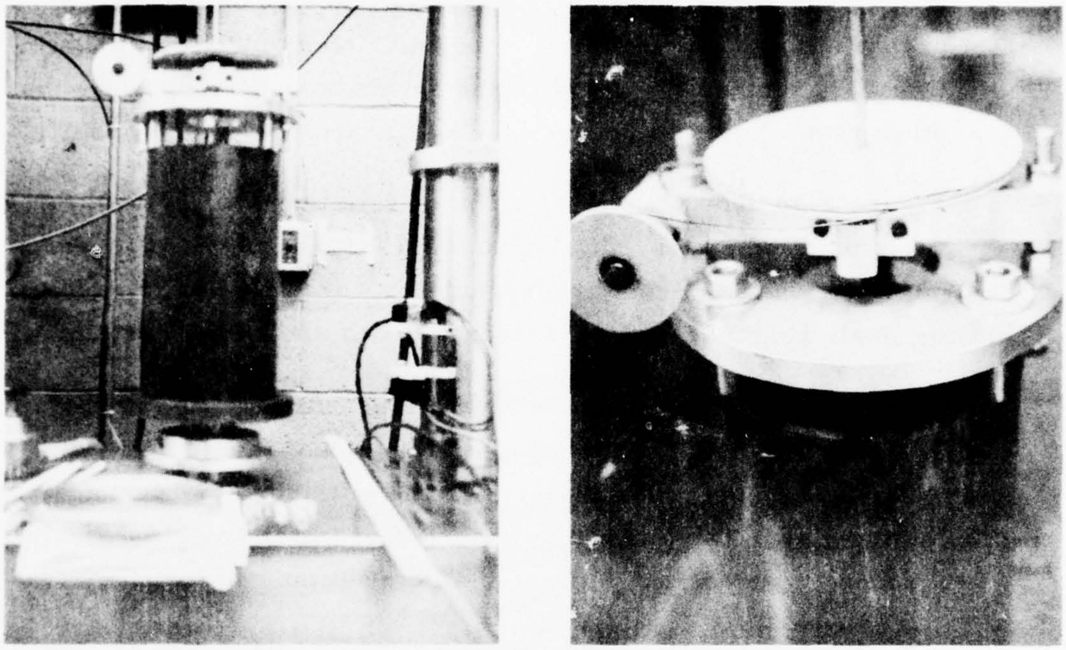


Figure 3-56. Laboratory Torque Vane Mounted on Sample Vibration Cylinder

# TABLE 3-16 - MTS Program Test Results

| SAMPLE NO. | LOCATION* IN CYLINDER | TYPE OF SOIL | AMPLITUDE AND FREQUENCY | INITIAL WATER CONTENT (% WEIGHT OF SOLIDS) | TIME OF SAMPLING (HRS) | FINAL WATER CONTENT (%) | | DRY DENSITY (SUSPENDED SOLIDS) (PCF) | | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | MTS | STATIC | MTS | STATIC | |
| S-1-A | Top | Silt | ± 2.5" At 0.2 Hz | At Liquid Limit | 0 | 54.5 | 54.0 | 64.7 | 67.1 | No apparent sedimentation during tests |
| S-1-B | Middle | | | | | 54.5 | 52.1 | 67.7 | 69.2 | |
| S-1-C | Bottom | | | | | 54.2 | 52.1 | 68.0 | 68.6 | |
| S-1-D | Top | | | | 7 | 53.1 | 53.3 | 68.0 | 67.7 | |
| S-1-E | Middle | | | | | 53.4 | 53.4 | 66.9 | 67.4 | |
| S-1-F | Bottom | | | | | 52.5 | 55.0 | 67.0 | 67.1 | |
| S-2-A | Top | Silt | ± .001" At 25 Hz | At Liquid Limit | 0 | 58.0 | 54.8 | 65.5 | 67.0 | No apparent sedimentation during tests |
| S-2-B | Middle | | | | | 55.7 | 56.7 | 66.3 | 66.0 | |
| S-2-C | Bottom | | | | | 56.5 | 58.2 | 66.1 | 65.3 | |
| S-2-D | Top | | | | 7 | 55.6 | 55.6 | 66.4 | 66.6 | |
| S-2-E | Middle | | | | | 55.7 | 56.1 | 66.4 | 66.1 | |
| S-2-F | Bottom | | | | | 56.3 | 57.3 | 66.1 | 65.6 | |
| S-3-A | Top | Silt | ± 2.5" At 0.2 Hz | At 2x Liquid Limit | 0 | 95.6 | 96.0 | 45.8 | 46.4 | No apparent sedimentation during tests |
| S-3-B | Middle | | | | | 95.2 | 96.6 | 45.6 | 45.4 | |
| S-3-C | Bottom | | | | | 95.7 | 96.6 | 45.6 | 46.3 | |
| S-3-D | Top | | | | 7 | 98.2 | 94.3 | 45.1 | 48.1 | |
| S-3-E | Middle | | | | | 97.2 | 94.3 | 46.7 | 47.1 | |
| S-3-F | Bottom | | | | | 99.3 | 95.7 | 46.2 | 45.9 | |
| S-4-A | Top | Silt | ± .001" At 25 Hz | At 2x Liquid Limit | 0 | 99.8 | 95.5 | 45.2 | 47.2 | No apparent sedimentation during tests |
| S-4-B | Middle | | | | | 98.9 | 96.2 | 46.4 | 47.0 | |
| S-4-C | Bottom | | | | | 96.2 | 97.4 | 47.7 | 46.7 | |
| S-4-D | Top | | | | 7 | 101.5 | 99.0 | 45.7 | 45.9 | |
| S-4-E | Middle | | | | | 100.3 | 99.2 | 46.2 | 45.9 | |
| S-4-F | Bottom | | | | | 101.0 | 100.5 | 46.4 | 46.8 | |
| S-5-A | Top | Silt | ± 2.5" At 0.2 Hz | At 8x Liquid Limit | 0 | 353.5 | 361.3 | 15.9 | 17.6 | $H_2O$/Silt interface 8½ inches from top after tests |
| S-5-B | Middle | | | | | 347.7 | 356.1 | 16.3 | 16.1 | |
| S-5-C | Bottom | | | | | 360.6 | 324.5 | 15.9 | 17.3 | |
| S-5-D | Top | | | | 7 | 465.0 | 455.4 | 12.5 | 12.8 | |
| S-5-E | Middle | | | | | 357.9 | 334.8 | 15.5 | 16.8 | |
| S-5-F | Bottom | | | | | 110.8 | 102.2 | 42.6 | 44.8 | |
| S-6-A | Top | Silt | ± .001" At 25 Hz | At 8x Liquid Limit | 0 | 365.8 | 388.3 | 15.5 | 14.4 | $H_2O$/Silt interface 8½ inches from top after tests |
| S-6-B | Middle | | | | | 367.9 | 364.6 | 15.4 | 15.4 | |
| S-6-C | Bottom | | | | | 379.0 | 368.4 | 15.0 | 15.1 | |
| S-6-D | Top | | | | 7 | 460.1 | 496.3 | 12.2 | 11.7 | |
| S-6-E | Middle | | | | | 363.8 | 360.3 | 15.3 | 15.4 | |
| S-6-F | Bottom | | | | | 99.0 | 117.3 | 45.0 | 40.0 | |
| S-7-A | Top | Silt | ± 2.5" At 0.2 Hz | At 5x Liquid Limit | 0 | 249.9 | 243.4 | 21.5 | 21.9 | $H_2O$/Silt interface 1½ inches from top after tests |
| S-7-B | Middle | | | | | 243.0 | 250.6 | 22.0 | 21.5 | |
| S-7-C | Bottom | | | | | 250.0 | 248.9 | 21.4 | 21.5 | |
| S-7-D | Top | | | | 7 | 260.0 | 278.5 | 20.6 | 19.5 | |
| S-7-E | Middle | | | | | 278.0 | 275.8 | 19.5 | 19.7 | |
| S-7-F | Bottom | | | | | 219.3 | 230.4 | 23.9 | 23.0 | |
| S-8-A | Top | Silt | ± .001" At 25 Hz | At 5x Liquid Limit | 0 | 260.9 | 259.0 | 21.1 | 21.3 | $H_2O$/Silt interface 1½ inches from top after tests |
| S-8-B | Middle | | | | | 260.3 | 258.8 | 21.3 | 21.5 | |
| S-8-C | Bottom | | | | | 262.0 | 258.5 | 21.1 | 21.5 | |
| S-8-D | Top | | | | 7 | 266.4 | 278.3 | 21.1 | 20.3 | |
| S-8-E | Middle | | | | | 262.1 | 263.4 | 21.4 | 20.8 | |
| S-8-F | Bottom | | | | | 226.4 | 242.0 | 23.7 | 23.0 | |
| S-17-A | Top | Silt | ± .001" At 60 Hz (Shaking table) | At 2x Liquid Limit | 0 | 109.2 | 112.2 | 42.7 | 42.3 | No apparent sedimentation during tests |
| S-17-B | Middle | | | | | 109.1 | 113.2 | 43.2 | 41.6 | |
| S-17-C | Bottom | | | | | 111.1 | 112.4 | 41.4 | 42.5 | |
| S-17-D | Top | | | | 7 | 103.3 | 102.6 | 44.9 | 44.5 | |
| S-17-E | Middle | | | | | 114.0 | 109.3 | 42.8 | 43.0 | |
| S-17-F | Bottom | | | | | 106.2 | 113.5 | 44.5 | 42.8 | |

*The surface of solids was considered as top of sample before and after sedimentation.

## TABLE 3-16 (cont.)

| SAMPLE NO. | LOCATION* IN CYLINDER | TYPE OF SOIL | AMPLITUDE AND FREQUENCY | INITIAL WATER CONTENT (% WEIGHT OF SOLIDS) | TIME OF SAMPLING (HRS) | FINAL WATER CONTENT (%) | | DRY DENSITY (SUSPENDED SOLIDS) (PCF) | | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | MTS | STATIC | MTS | STATIC | |
| C-9-A | Top | Clay | ± 2.5" At 0.2 Hz | At Liquid Limit | 0 | 86.3 | 82.8 | 49.1 | 49.0 | No apparent sedimentation during tests |
| C-9-B | Middle | | | | | 86.7 | 81.5 | 50.0 | 50.8 | |
| C-9-C | Bottom | | | | | 87.9 | 81.3 | 49.1 | 50.1 | |
| C-9-D | Top | | | | 7 | 87.4 | 83.5 | 50.0 | 50.4 | |
| C-9-E | Middle | | | | | 87.1 | 83.3 | 50.4 | 50.7 | |
| C-9-F | Bottom | | | | | 85.1 | 84.4 | 50.6 | 50.8 | |
| C-10-A | Top | Clay | ± .001" At 25 Hz | At Liquid Limit | 0 | 90.3 | 92.2 | 48.5 | 48.7 | No apparent sedimentation during tests |
| C-10-B | Middle | | | | | 88.0 | 92.8 | 50.1 | 48.0 | |
| C-10-C | Bottom | | | | | 87.5 | 90.6 | 50.4 | 49.1 | |
| C-10-D | Top | | | | 7 | 89.0 | 92.7 | 49.7 | 49.2 | |
| C-10-E | Middle | | | | | 92.0 | 85.9 | 49.0 | 50.4 | |
| C-10-F | Bottom | | | | | 91.6 | 88.5 | 49.4 | 50.3 | |
| C-11-A | Top | Clay | ± 2.5" At 0.2 Hz | At 2x Liquid Limit | 0 | 134.1 | 139.6 | 35.2 | 34.6 | No apparent sedimentation during tests |
| C-11-B | Middle | | | | | 142.7 | 142.5 | 35.4 | 35.9 | |
| C-11-C | Bottom | | | | | 142.1 | 141.2 | 35.9 | 35.2 | |
| C-11-D | Top | | | | 7 | 156.3 | 149.7 | 31.8 | 33.2 | |
| C-11-E | Middle | | | | | 174.1 | 180.0 | 29.7 | 28.9 | |
| C-11-F | Bottom | | | | | 171.8 | 225.6 | 30.4 | 24.1 | |
| C-12-A | Top | Clay | ± .001" At 25 Hz | At 2x Liquid Limit | 0 | 158.2 | 160.6 | 30.9 | 31.2 | No apparent sedimentation during tests |
| C-12-B | Middle | | | | | 156.4 | 157.5 | 31.9 | 31.6 | |
| C-12-C | Bottom | | | | | 157.6 | 158.3 | 31.9 | 31.6 | |
| C-12-D | Top | | | | 7 | 168.4 | 159.6 | 29.8 | 30.9 | |
| C-12-E | Middle | | | | | 162.2 | 162.2 | 30.7 | 30.6 | |
| C-12-F | Bottom | | | | | 169.5 | 162.6 | 29.5 | 30.7 | |
| C-13-A | Top | Clay | ± 2.5" At 0.2 Hz | At 8x Liquid Limit | 0 | 440.9 | 468.1 | 12.8 | 12.3 | $H_2O$/Clay interface 7½ inches from top after MTS test and 8½ inches from top after Static test |
| C-13-B | Middle | | | | | 445.4 | 438.1 | 13.0 | 13.4 | |
| C-13-C | Bottom | | | | | 353.4 | 454.9 | 16.3 | 12.7 | |
| C-13-D | Top | | | | 7 | 445.2 | 531.3 | 12.8 | 10.9 | |
| C-13-E | Middle | | | | | 309.5 | 384.1 | 17.7 | 15.0 | |
| C-13-F | Bottom | | | | | 117.3 | 138.0 | 40.4 | 35.5 | |
| C-14-A | Top | Clay | ± .001" At 25 Hz | At 8x Liquid Limit | 0 | 723.0 | 720.6 | 8.4 | 8.4 | $H_2O$/Clay interface 9 inches from top after tests |
| C-14-B | Middle | | | | | 727.3 | 718.2 | 8.2 | 8.5 | |
| C-14-C | Bottom | | | | | 732.4 | 735.5 | 8.2 | 8.3 | |
| C-14-D | Top | | | | 7 | 458.4 | 473.7 | 12.4 | 11.9 | |
| C-14-E | Middle | | | | | 394.0 | 389.2 | 14.3 | 14.6 | |
| C-14-F | Bottom | | | | | 350.7 | 337.5 | 15.9 | 16.7 | |
| C-15-A | Top | Clay | ± 2.5" At 0.2 Hz | At 5x Liquid Limit | 0 | 387.6 | 372.8 | 14.4 | 14.8 | $H_2O$/Clay interface 1 inch from top after tests |
| C-15-B | Middle | | | | | 381.3 | 398.0 | 14.7 | 14.2 | |
| C-15-C | Bottom | | | | | 387.9 | 394.9 | 14.4 | 14.1 | |
| C-15-D | Top | | | | 7 | 412.9 | 406.0 | 13.8 | 13.8 | |
| C-15-E | Middle | | | | | 385.8 | 409.9 | 14.8 | 13.7 | |
| C-15-F | Bottom | | | | | 369.8 | 381.6 | 15.5 | 14.6 | |
| C-16-A | Top | Clay | ± .001 At 25 Hz | At 5x Liquid Limit | 0 | 383.2 | 382.7 | 14.6 | 14.4 | $H_2O$/Clay interface 1½ inches from top after tests |
| C-16-B | Middle | | | | | 374.9 | 378.8 | 14.9 | 14.8 | |
| C-16-C | Bottom | | | | | 389.5 | 382.3 | 14.4 | 14.6 | |
| C-16-D | Top | | | | 7 | 379.3 | 421.3 | 14.7 | 13.2 | |
| C-16-E | Middle | | | | | 374.1 | 389.1 | 14.7 | 14.2 | |
| C-16-F | Bottom | | | | | 341.5 | 368.0 | 16.3 | 15.1 | |
| C-18-A | Top | Clay | ± .001" At 60 Hz (Shaking table) | At 2x Liquid Limit | 0 | 166.5 | 165.3 | 31.4 | 32.0 | No apparent sedimentation during tests |
| C-18-B | Middle | | | | | 173.3 | 169.2 | 29.4 | 30.6 | |
| C-18-C | Bottom | | | | | 170.0 | 163.3 | 31.7 | 30.8 | |
| C-18-D | Top | | | | 7 | 176.9 | 173.0 | 28.5 | 29.4 | |
| C-18-E | Middle | | | | | 168.4 | 156.7 | 30.2 | 31.5 | |
| C-18-F | Bottom | | | | | 164.9 | 158.0 | 30.3 | 31.3 | |

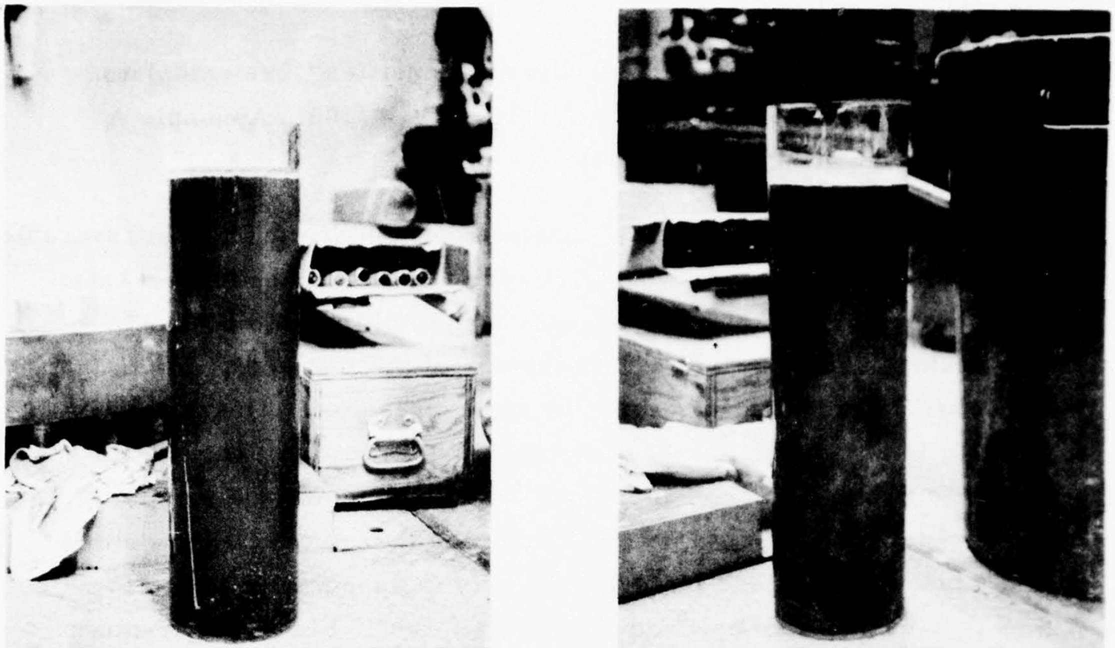
*The surface of solids was considered as top of sample before and after sedimentation.

weight and sample volume; the dry densities were calculated by dividing the wet density by: 1 + PCM/100. Appendix A lists additional background.

The MTS program results indicate that only time had a measurable effect on the dredged material and then only at the higher water contents. Changes in frequency and amplitude did not seem to make any difference in the behavior of the material. In fact, none of the vibrations tried had any apparent effect. The results of the static and dynamic tests are practically identical, considering the order of accuracy of the sampling technique and the sample preparation. Only in a few cases was there noted a very slight tendency for more materials to stay in suspension during the dynamic test as opposed to the static test. Figure 3-57 shows the effect at 5 times the liquid limit.

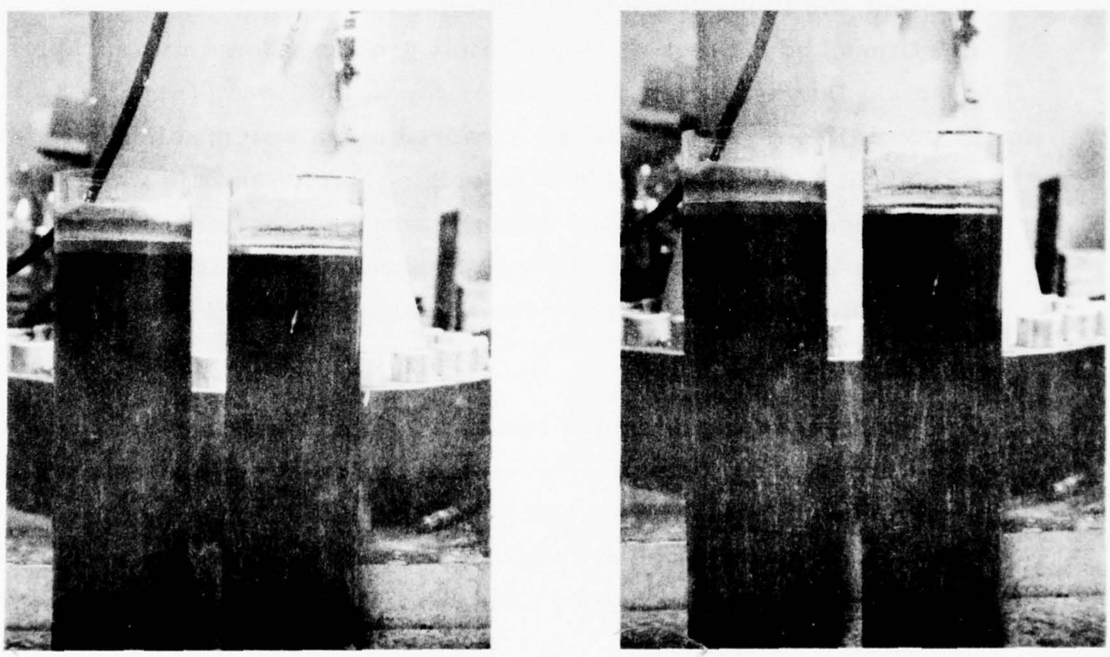
Generally it can be concluded that no measurable time and vibration effects were noted during the tests of both silts and clays at either the liquid limit or at twice the liquid limit. The tests at 5 times and 8 times the liquid limit produced some measurable effects. During the tests at 5 times the liquid limit (Tests 7, 8, 15 and 16) some sedimentation occurred and a water/solids interface about 1-1/2 inches below the original top of sample was established. As a result the bottom samples experienced a decrease in water content of about 40 percent moisture with an accompanying increase in dry density ranging from 2 to 4 pcf.

The sedimentation during the tests at 8 times the liquid limit was more pronounced. The water/solids interface after the tests were established at 8-1/2 inches below the original surface for the silts (Tests 5 and 6) and at 7-1/2 to 9 inches below the original surface for the clays (Tests 13 and 14). Figures 3-58 through 3-61



5 x LL after 7 hrs static
low amplitude - 25 Hz

5 x LL after 7 hrs MTS
low amplitude 25 Hz



5 x LL after 7 hrs static & MTS
high amplitude - 0.2 Hz

Figure 3-57. MTS and Static Column at Five Times the Liquid Limit

present the results of eight times the liquid limit (8 x LL) tests graphically. The water content of top, middle and bottom samples are plotted against depth within the cylinder for both dynamic and static tests.

The figures show no apparent difference in the physical properties of the samples between the dynamic and static test results. Figures 3-58 and 3-59 show that as the silt settled to the bottom of the cylinders, the water content increased at the top and decreased at the bottom, thus creating a water content gradient, and corresponding density gradient, with depth. For the clay the result is similar although not as pronounced. During Test 13 (Figure 3-60) the initial and final properties followed the trend of the silt samples. The initial water content turned out to be well below that assigned for 8 x LL of clay but was very close to 8 x LL of silt. The series of samples behaved like the silt, and it is possible that due to some inconsistency in the sample preparation a silt pocket may have existed in the mixing container, which was inadvertently sampled for Test 13. The initial values of water content in Test 14 (Figure 3-61) are in the range which was assigned for 8 x LL for clays. Here the trend of the water content and density gradient is repeated but to a lesser degree, indicating that the clay particles tend to stay in suspension longer than silt particles.

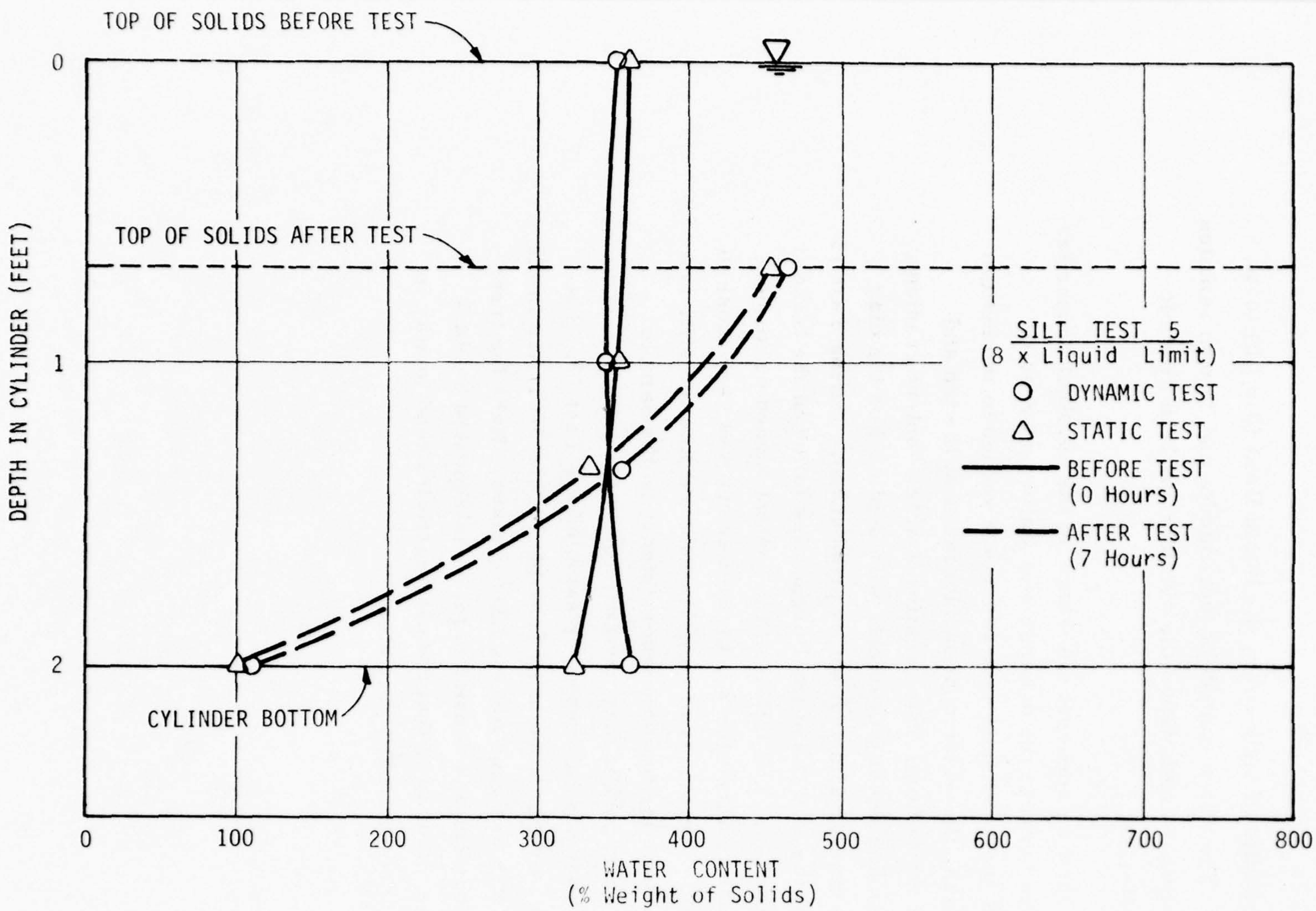



Figure 3-58. MTS PROGRAM RESULTS

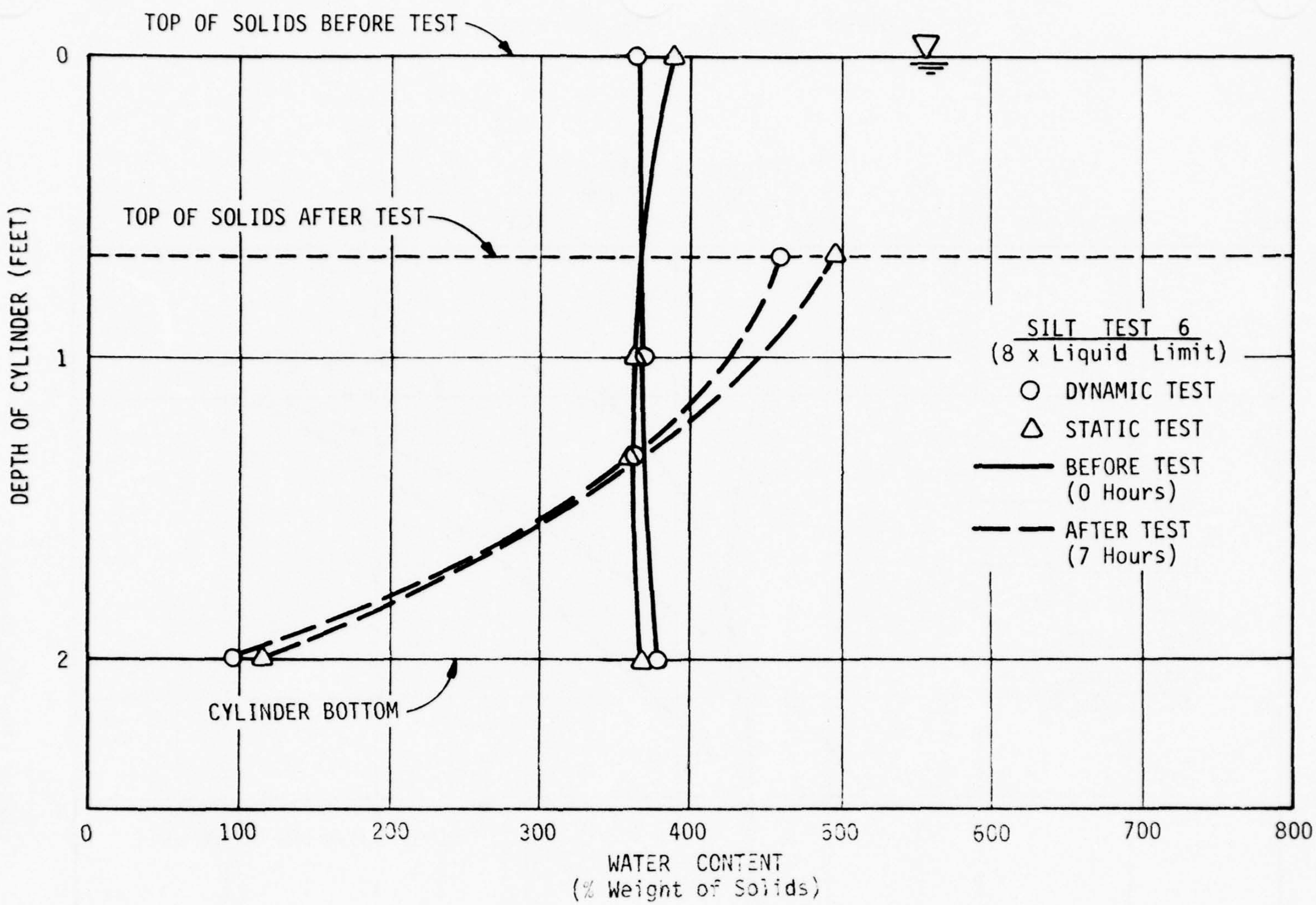



Figure 3-59. MTS PROGRAM RESULTS

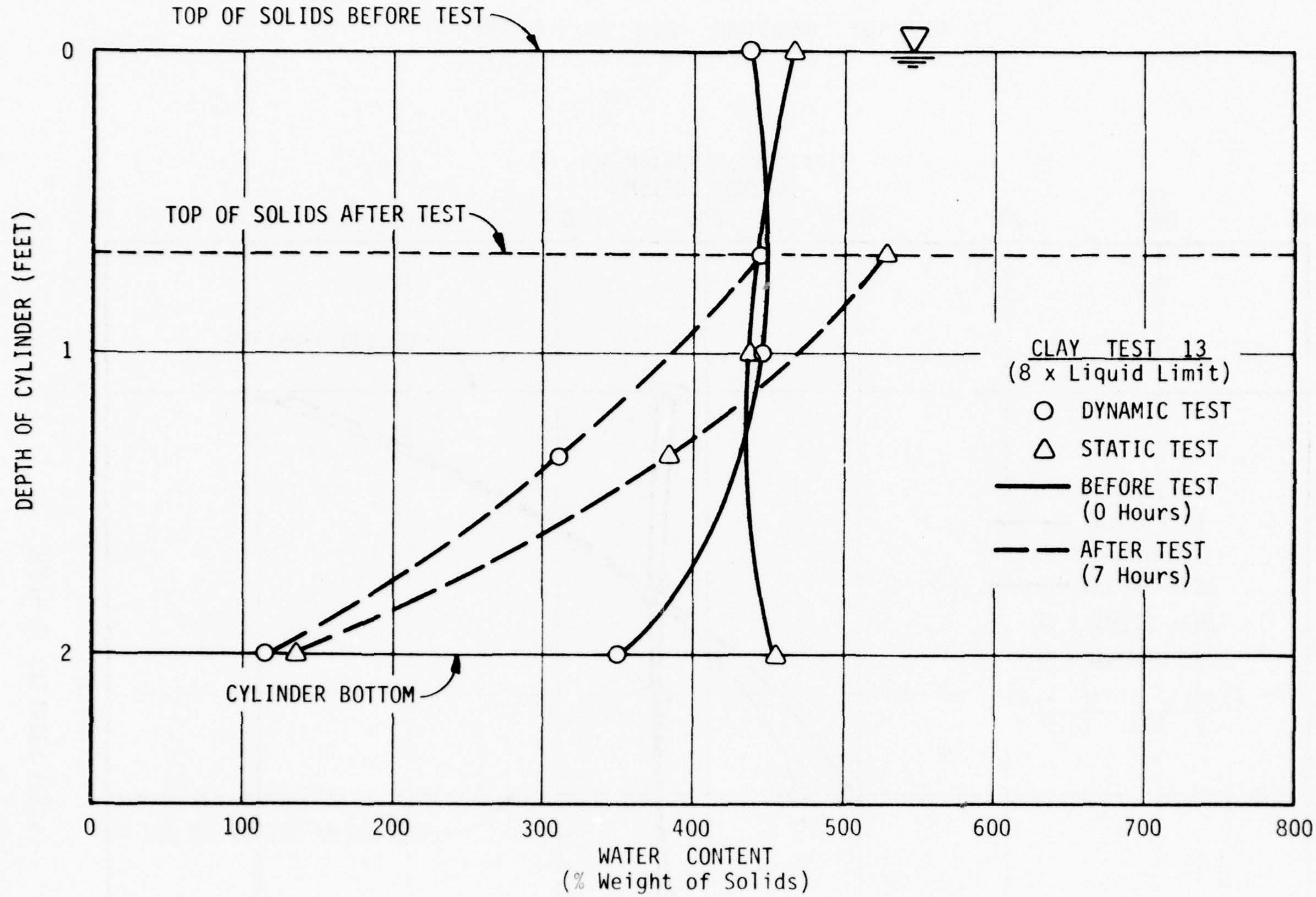



Figure 3-60. MTS PROGRAM RESULTS

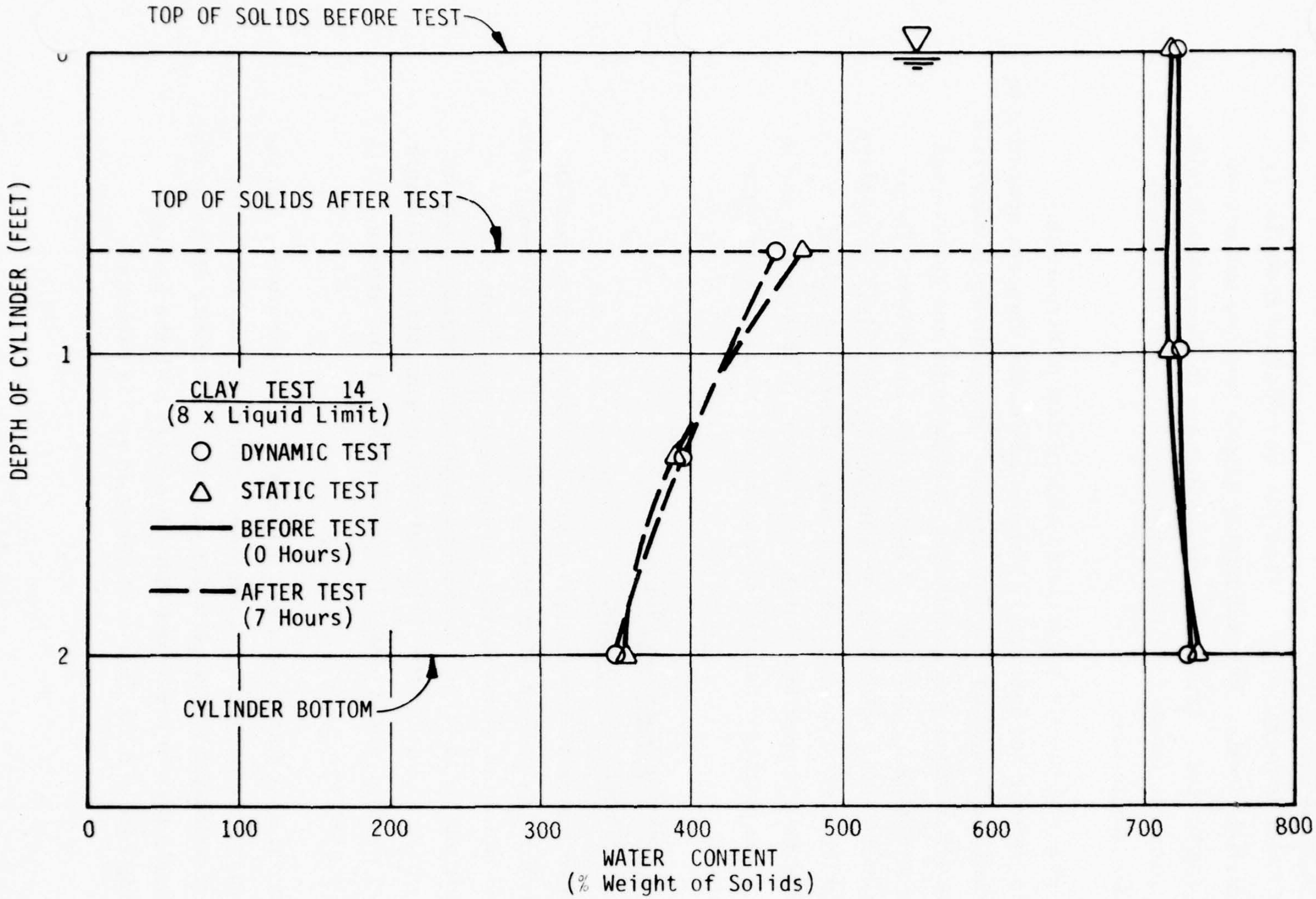



Figure 3-61. MTS PROGRAM RESULTS

## E. Disposal Phase

### 1. General Discussion

Open water disposal of San Francisco Bay dredged material is accomplished using bottom dumping hopper dredges and bottom dumping barges. Typical disposal sites are in Carquinez Straits, near Alcatraz Island, the San Francisco Bay, and offshore at the 100 fathom line.

This phase of the study consisted of a review of dispersion considerations and laboratory simulations of the disposal operation to examine the mounding and dispersion of typical dredged materials. These simulations were conducted in aquariums and glass-sided tanks at depths of 1.3 feet, 2 feet, 4 feet, and 9 feet. Drops were made using "silt" from Pinole Shoals and "clay" from Mare Island Straits. Two disposal container shapes were used, simulating a hopper and a clamshell barge, in fresh water and in salt. The primary material characteristic varied was percent moisture.

### 2. Dispersion Considerations

#### a. Transport mechanisms.

Dredged material may consist of any substances which exist in the sediments at the bottom of waterways. The major components will be solid inorganic particles ranging in size from molecular dimensions up to large rocks and boulders, an organic fraction, and water. Additionally, relatively minor amounts of pollutants (heavy metals, pesticides, algal nutrients, oil and grease, etc.) will be present.

The most important factors in dispersion considerations, and the subsequent potential for erosion and resuspension, will be the size distribution of the particles and the bulk density of the dredged material. Size is important since it affects both the particle settling rate and the degree of cohesion among particles.

Bulk density will determine the rate of descent immediately following the dump and is a function of the density of individual particles, the water content, and the proportion of lighter materials such as organics. Water content and cohesion affect the behavior of the dumped material during the descent and collapse phase by controlling the growth of the cloud and the spreading rate during the collapse phase.

Clark, et al [8] describe the total transport by dividing it into four basic transport phases:

(1) Convective descent

(2) Collapse

(3) Long term dispersion

(4) Bottom transport and resuspension

(1) Convective descent. For this discussion dredged materials are assumed to be dumped essentially instantaneously as from a hopper dredge or a dump barge. Under these conditions the dumped material will possess an initial downward momentum and a density greater than that of the surrounding water. These result in forces that cause the material to settle in the form of a cloud, or density current, rather than as individual particles. As the cloud settles, shear stresses are developed at the interface between the moving cloud and the ambient water. These stresses result in dissipation of the initial momentum and in the creation of turbulent eddies that will entrain ambient fluid. In the case of clouds possessing an initial momentum, at the time of release vortex rings will form and will tend to cause deeper penetration of the ambient water.

The convective descent phase with dredged materials occurs very rapidly. Based on observations at the New Haven, Connecticut

dump site Gordon [ 9 ] estimated the convective descent velocity for barge dumped material to be one foot per second in 60 feet of water. Marine silt having a high water content was dumped from a scow and measurements of cloud velocity were made with a transmissometer at known distances from the discharge point.

Observations, also by Gordon [ 10] , of another dump at the New Haven site again showed very rapid descent. In this case 2300 cubic yards of dredged material contained 66 percent water. The solid portion was 15 percent sand, 60 percent silt, and 25 percent clay. The water depth at the site was 51 feet. It is not known whether the transmissometer observation of the passage of the cloud occurred during the convective descent phase or after the cloud had impacted on the bottom with subsequent horizontal transport across the bottom. If it could be assumed that the cloud descended directly onto the transmissometer, then the descent velocity would be 2.4 feet per second. If on the other hand, the transmissometer observation was of the spreading cloud after bottom contact, then the convective descent velocity must have been even greater. The important point is that dumping materials in relatively shallow water produces a rapid convective descent of the material with a vertical velocity of at least one foot per second and possibly much greater. Settling velocities calculated for individual particles do not apply during this form of transport. The time during which the cloud is in contact with the upper portions of the water column are in the order of a minute or less so that ambient water currents, except near the bottom, are of little consequence in dredged material placement, except as they affect the transport of any turbidity cloud that may be generated during the descent.

(2) Collapse. The second phase of transport occurs when the cloud begins a dynamic vertical collapse characterized by horizontal spreading. Collapse is driven primarily by a pressure force and resisted by inertial and frictional forces.

Dynamic collapse will occur when the cloud encounters a boundary, either a pycnocline or the ocean bottom. In the case of dumping of dredged materials it is important that, if a pycnocline exists, the cloud will penetrate the layer and reach the ocean bottom. An indication of whether penetration will occur is given by an expression developed by Sullivan and discussed by Brooks [ 11 ] . Based on dimensional analysis and small scale laboratory experiments Sullivan developed the following equation:

$$\Lambda = \frac{(\rho_2 - \rho_1) Z^3}{(\rho_i - \rho_1) V}$$

where $\Lambda$ = dimensionless parameter

$\rho_2$ = density of lower layer

$\rho_1$ = density of upper layer

Z = depth from release point of interface

$\rho_i$ = initial density of heavy fluid injected

V volume of heavy fluid injected

The following criteria were determined experimentally: $\Lambda > 29$ less than 10 percent of injected slug penetrates the lower layer. $\Lambda < 1.5$ more than 90 percent of injected slug continues into the lower layer.

Brooks cautions against complete acceptance of Sullivan's criteria because the experiments were conducted at low Reynolds number for which the flow was partially laminar. However, several

tentative conclusions can be drawn. First, for dredged materials dumped in shallow water, penetration of a pycnocline is very likely. Second, if problems are to occur, they will probably happen in deeper water since the value of $\Lambda$ increases as the third power of the interface depth. Third, to maximize the likelihood of interface penetration, the volume of material dumped should be maximized.

Releases of large quantities of dredged materials in shallow water will usually penetrate a density layer and impact on the ocean bottom. The material will either mound or the cloud will flatten out and appear somewhat like a "pancake" as it assumes a horizontal circular shape (assuming a flat bottom and no obstructions) with a small vertical dimension. Under these conditions, flow may occur in the form of a density or turbidity current.

While the bottom turbid cloud is spreading it is also settling. Gordon estimated that at least 80 percent of the dumped material was deposited within a radius of 100 feet and that 90 percent was within a radius of about 400 feet. Other researchers have also observed the spread of dredged materials during the collapse phase. Sustar and Ecker [12] observed four distinguishable layers in the water column following dredged material disposal from a hopper dredge. The upper portion of the water column extending from 25 to 35 feet below the surface was unaffected by the dumping operation. This is approximately the draft of a hopper dredge. A turbid layer existed from 3 to 15 feet above the bottom. The depth and sediment concentration in the turbid layer depended on current velocities, tide and sea states. When currents and sea state increased, the depth of the layer and concentration of sediment in the layer increased. On the bottom was a layer of

fluid sediment three to six inches deep overlying a compacted sediment bottom.

Turbidity or density flows of sediments released from dredging operations have often been observed. May [ 13 ] has reported on open water disposal from channel and shell dredging by hydraulic dredges. Almost all the sediment settled very rapidly and was transported along the bottom as a separate flocculated density layer. The sediment which was not deposited immediately under the dredge was transported in the density flow. Concentrations of 10,000 mg/l were found within 400 feet of the discharge point and concentration levels over 1000 mg/l extended out at least 1800 feet.

Masch and Espey [ 14 ] observed the continuous discharge from a hydraulic shell dredge. The sediments were classified as a silty-clay. Data obtained from bottom funnel traps compared with others at three foot elevation showed that the sediment moved as a density current. Sediment concentrations six inches from the bottom were as high as 150,000 mg/l, and as the distance from the dredge increased the thickness of the layer decreased although the concentration remained high. The density current was well defined even at distance greater than 1500 feet. At a distance 600 feet from the dredge the density current had concentrations greater than 100,000 mg/l, was about eight inches thick and was over 1200 feet wide. While these observations, and those of May, pertain to continuous hydraulic dredging in shallow water, rather than instantaneous discharges from hopper dredges or barges, it is clear that under certain conditions dredged materials can be transported great distances in a density current.

Probably the first investigators to study in detail the movement of clay suspensions in water were Einstein and Krone [ 15 ] who

reported on laboratory observations concerning cohesive sediment transport of San Francisco Bay muds. The sediments contained more than half clay with the remainder essentially all less than $100\mu$ in diameter. Flocculation was considered to be one of the most important factors in cohesive sediment transport. Naturally occurring flocculation is an electrochemical process associated with clay particles. In all waters, and particularly in salt water, flocculation results in particles grouping together due to inter-particle contacts. These larger particles then settle faster than would individual particles. The important characteristic of the particles for this phenomena to take place is that the particles must be cohesive. This in turn depends to a large extent on particle size, with clay particles ( $<$ 0.002 mm diameter) exhibiting a large measure of cohesiveness, silts (0.002 mm to 0.06 mm) having some cohesiveness, and sand and gravel ( $>$ 0.06 mm) possessing none. In addition to cohesiveness, other factors affecting flocculation are the type and concentration of particles, the nature and concentration of dissolved salts, or the level of mixing energy present. Low flocculation mixing energy would not produce enough interparticle contacts and too high a level of mixing would shear apart particles previously driven together. Among the conclusions drawn by Einstein and Krone were:

a. When sediment concentration exceeded about 10 gm/l in salt water flocculation occurred rapidly and during settling an interface existed between what was termed "fluid mud" and relatively clear water above it.

b. The concentration at which the settling fluid mud becomes too great to readily allow flow (the onset of consolidation) was 167 gm/l.

c. Fluid muds behave as Bingham fluids, that is they behave as true liquids when the shear stress is above a critical value and as

solids below that value.

d. Fluid muds can be transported by gravity flow provided that the bottom slope is sufficiently steep to start and maintain flow. Once started, a gravity flow of fluid mud could be maintained on a flatter slope than that required to get it started.

e. The flow of fluid muds probably is not affected by bulk flow of the overlying water.

f. At suspended sediment concentrations less than 300 mg/l very little flocculation will occur, but increases in mixing energy can increase flocculation even at this low solids concentration.

White [16] has conducted laboratory studies in a flume to investigate the movement of dredge sediments as a density current. Using sediments from shell dredging areas in Galveston Bay, Texas, containing 50 percent clay with lesser amount of silt, sand, and organic matter he observed that:

a. Both currents and sloping bottoms tend to increase the movement of a fluid mud, but the effect of the current is much less than that of the sloping bottom indicating that gravity is the predominant force in the movement of density layers.

b. When a shallow dike was placed across the flume with an opening at one end, the layer flowed past the dike through the opening. If the dike were placed completely across the flume the layer fell back until it increased in height due to the introduction of additional material and then flowed over the dike. Appreciable deposition occurred in front of the dike while the layer's progress was impeded by the dike.

c. A suspended sediment concentration of 2,200 mg/l was required to initiate density layer flow.

d. Under flume conditions of no flow and a 1 deg slope, the mud flow interface front was observed to move at a velocity of 2.2 feet per minute.

e. Strong currents tend to prohibit the formation of density currents by turbulent mixing and sweeping the sediment away before they can build up to sufficient concentration for layer development.

As a result of their laboratory and field work (partly based on White's work) Masch and Espey [ 14 ] concluded that for a sediment layer to form sediment concentrations greater than 10 gm/l are required so that settling of the layer is hindered and the layer can remain in a fluid form. It was estimated that the mud layer must contain more than 80 percent by weight of particles smaller than 0.0625 mm (the upper end of the silt size range) of which about 50 percent or more should be in the clay range. Even though settling is hindered in the mud density current, consolidation of the flocculated particles takes place and the layer gains shear strength. Within a few days the layer usually has enough strength to resist the shear produced by low magnitude tidal currents and is no longer capable of being moved except by higher energy currents which may cause erosion and resuspension.

In summary, cohesive dredged materials will flow as a density current under conditions of solids concentrations between 10 and 170 gm/l, clay content greater than about 50 percent, and the presence of a driving force due to a slope or to a hydraulic head. The major force producing motion is gravity rather than currents.

(3) Long term dispersion. Long-term dispersion refers to mixing processes which occur after the convective descent and collapse phases have been completed, and include eddy diffusion due to random currents, mixing by wind waves, and mixing by currents

traveling essentially in one direction such as tidal currents. Each of these is a complicated phenomena and all may be acting on the water column at the same time making an estimate of the final fate of suspended dredged materials very difficult.

At the completion of the collapse phase suspended solids subject to dispersion will be of two fundamentally different types. One is those solids which are part of the density current remaining in suspension within perhaps a foot of the bottom. They will be in highly concentrated form and may possess some shear strength to resist transport by currents. Experiments by Einstein and Krone [ 15 ] and White [ 16 ] have shown that the upper concentration limit on a sediment suspension beyond which further density flow will not occur is the range of 170 gm/l. These particles will continue to settle, but at a slow rate due to bridging of particles and the difficulty of voiding water trapped among the particles. By observing sediments settling in a laboratory cylinder Einstein and Krone estimated that the fluid mud phase lasted only about two hours in still water after which bulk flow is no longer possible and consolidation is taking place. Concentrations of sediments which can resist a given shear by flowing water are shown in Table 3-17.

TABLE 3-17

Sediment Resistance to Bulk Flow by Shear [15]

| Average Velocity, cm/sec | Shear Strength, dynes/cm$^2$ | Sediment Concentration g/l |
|---|---|---|
| 30 | 0.98 | 17 |
| 60 | 3.43 | 59 |
| 90 | 7.37 | 127 |
| 120 | 12.6 | 217 |

This data indicates that once consolidation has begun, the fluid mud can continue consolidating at flow velocities higher than those generally to be encountered.

The second type of solids subject to long term dispersion are those remaining in the water column as a result of the dumping operation. As the cloud rapidly settles and spreads across the ocean bottom eddies will spin off and carry solids out of the main cloud. The concentration of solids remaining in the water column will be sufficiently low so that the water will not have sufficient excess density to readily sink to the bottom and rejoin the main cloud. Gordon in his observations at the New Haven dump site [9] found that a residual drifting cloud existed after the dump. Turbidity profiles defined the cloud to be approximately 30 feet thick and have a diameter of about 200 feet. Measurements of the solids content of the cloud indicated that the total amount of solids contained in the cloud amounted to about one percent of the material dumped.

In summary, particles remaining in the density current are not likely to be dispersed by ambient current and will consolidate at the point where the density current stops. Particles remaining in the water column after the dump probably represent a very small portion of all materials dumped and are likely to be swept out of the dump site before they settle to the bottom even if the local current is quite small.

b. Dispersion Models

A search of the literature to review models related to dumping of dredged materials found very few such models. The following four models were identified and considered as possibly having application to the short-term dumping of dredged materials in a relatively shallow ocean environment.

Edge-Dysart model

B.L. Edge and B.C. Dysart of Clemson University have developed a mathematical model of barged material dispersion which is

composed of a combination of jet theory and sedimentation [ 17 ]. In the first part, a negatively buoyant jet discharged downward into a stratified environment is simulated. The second portion describes transport of material from the end of the jet to the floor of the ocean. The model assumes that waste material is pumped from a circular outlet at some distance below the moving barge. Assumptions made concerning the jet flow include:

a. Steady flow
b. Incompressible flow
c. Fully turbulent jet
d. Longitudinal turbulent transport is less than convective transport
e. Constant fluid properties

A number of factors limit the applicability of this model to barge and hopper dredge dumping. First, the Edge-Dysart model does not describe short time interval dumping. Second, for pump-out type dumping another model (the Koh-Chang model discussed later) was considered to be a better representation of that type of discharge. Finally, the model does not consider dynamic collapse.

MIT model. A three dimensional analytical model has been developed by Christodoulou, Leimkuhler, and Ippen at the Massachusetts Institute of Technology for the dispersion of fine suspended sediments in coastal waters [ 18 ]. The model has been adapted to computer solution and has undergone some field work in connection with the NOMES project. The authors have conducted a number of test computer runs to investigate the effect on model predictions of data inputs representative of conditions in Massachusetts Bay.

The MIT model is based on long term diffusion with consideration given to settling of solids and an ambient velocity field consisting of both long shore current and a tidal component. The sediments

are assumed to be introduced into the water at a constant rate from a uniform vertical line source. The line source is considered to be far enough from the shore that land-sea boundary effects do not occur. Water depth is assumed to be constant. Since consideration of individual dumping events is not within the scope of the model, but rather, it is intended to predict the long term fate of a dredged material plume such as that resulting from a sand mining operation, the model is not applicable to bottom dumping.

Koh-Chang Model

An extensive mathematical, computerized model [ 19 ] has been developed by R.C.Y. Koh and Y.C. Chang which considers the dispersion and settling of barged wastes disposed of in the ocean. The model considers three phases of dispersion, namely convective descent, dynamic collapse of the descending plume, and long-term diffusion.

The model assumes that the cloud of material will descend as a result of its initial velocity and negative buoyancy. As it descends, it will push the ambient water around it, experience a drag force, and entrain surrounding water. Solid particles in the cloud will tend to settle out if the water is deep enough. The convective descent phase will cease either by encounter with the ocean bottom, or by reaching a level at which the ambient water density changes rapidly, effectively reducing the negative buoyancy. Provided that the water is deep enough, a horizontal spreading will result as the cloud seeks a hydrostatic equilibrium with the ambient water. This effect has been termed the dynamic collapse. Following dynamic collapse, the plume will be dynamically passive and affected only by turbulent diffusion, advection, and settling out of solid particles. Long-term diffusion considerations start from the general non-steady state, three dimensional conservation of mass equation. The

dynamic collapse is the starting point for the long-term diffusion. Details of the mathematics and computer techniques employed to arrive at solutions to these expressions are complex and have been documented in the reference.

Krishnappan Model

B. B. Krishnappan of the Canadian Center for Inland Waters has developed a mathematical model [ 20 ] based upon experiments on the spreading rates of solid particles moving in a liquid medium. Earlier models, such as the Koh-Chang, assumed that the dredged material consisted of a liquid medium whose density is equal to the equivalent density of the dredged material. Krishnappan's laboratory experiments indicate that the behavior of the solid particle cloud is very different from that of a liquid cloud and the difference is a function of particle size. As the particle size decreases, the difference between the behavior of solid and liquid particle clouds also decreases, tending to zero in the limit.

Krishnappan's laboratory experiments showed that when a slug of uniform size particles is released in a homogeneous and stationary body of water, with zero initial velocity, the particles moved as a cloud with two distinct boundaries. The size of the cloud increased for some period and then maintained its size. Similarly, the velocity decreased until it reached a constant value equal to the settling velocity of individual particles.

The first part of the descent was called the initial phase, where the cloud size grew due to entrainment, and the second was called the settling phase in which the horizontal cloud size remained relatively constant and the descent velocity equalled the fall velocity of the individual solid particles.

Using laboratory experiments, Krishnappan established coefficients to define the cloud growth during the initial phase and the settling

phase. By superposition he was able to allow for large particles settling out of the cloud while still allowing the remainder of the cloud to continue the initial phase (entrainment). Based on this experimental approach he was able to develop equations to predict the horizontal size of the cloud, the vertical descent velocity, and the height of mounding on the bottom. With the equations for a number of typical cases, he concluded that the settling phase will only occur for small dumps or in deep water (thousands of meters).

Krishnappan's method is interesting in that it allows large particles to fall out vertically while still allowing the entrainment phase to go on. It is also appealing for its simplicity, not requiring a computer to perform the calculation. A shortcoming of the model is that it does not provide for a dynamic collapse.

## 3. Laboratory Simulation of Bottom Dumping

### a. Introduction

The intent of the disposal simulation, as described in the original scope of work, was to provide data that would allow qualitative predictions (trends) of the effects that physical changes in the dredged material would have on the dispersion when dumped in open water.

The physical changes of interest in the dredged material were examined in the dredging and transport phases of this study. This section examines the effect of these changes on the disposal phase.

There were two discrete sets of disposal simulations conducted. The first set, conducted in large, glass-sided tanks (see Figure 3-62) investigated the general effects that physical changes in dredged material had on descent, impact , bottom mounding, and bottom flow. The second set was conducted in small aquariums to investigate the relationship between percent moisture (PCM) and mounding under the dump point.

Figure 3-63 shows the test matrix chosen for the large tanks. The baseline test consisted of fresh water, a hopper disposal container, 4 foot water depth, 200 percent moisture material and 10 liters volume. All variations were around this baseline, and only one variable was changed at a time (e.g., baseline but volume of 1 liter). The matrix was repeated for both clay and silt.

The large tank disposal simulation utilized two sediments and four different percent moisture contents approximately covering the range of values produced by the three dredge types. Clamshell dredges are represented by the 100 percent moisture tests, hydraulic suction dredges by the 500 percent moisture tests, and hopper dredges by the intermediate values. Laboratory

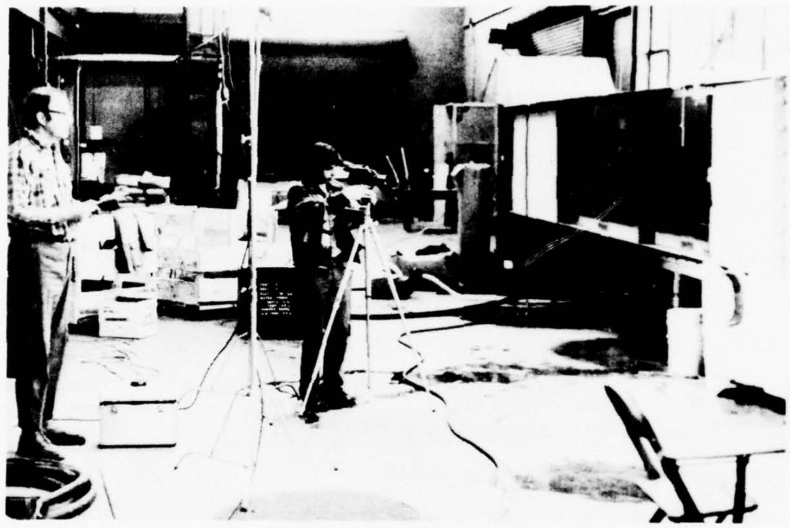


Figure 3-62. 4 foot x 4 foot x 40 foot Glass Tank

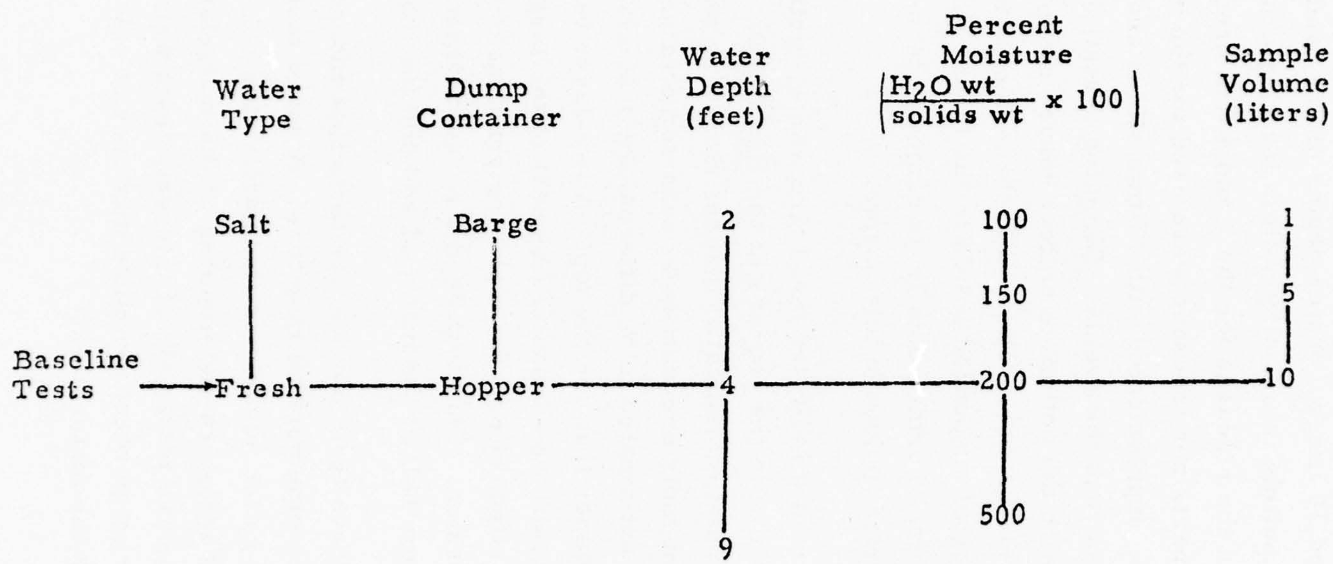



Figure 3-63. Test Matrix for Large Tanks

releases were made from containers simulating the hopper dredge and the split-hull barge in a two-dimensional cross-section as shown in Figures 3-64 and 3-65. Sediment volumes released were of 1, 5, and 10 liters. Water depths of 2, 4, and 9 feet were used and a compatible (i.e., clay bottom for clay drops) sediment bottom received the releases for the 2 and 4 foot drops. Salinities of zero and 30 parts per thousand were used for the water types. Super 8 movies, supplemented with 35 mm slides and black and white photos recorded the events. Sampling (Petri) dishes were inserted flush with the bottom to collect settled material and water samples of the plume were collected at three depths to determine the concentration of suspended solids in the turbid cloud moving across the bottom. Further details on sampling techniques are given at appropriate points in this chapter.

Our field test results indicated that there was a significant gradient in percent moisture in the hoppers of the HARDING. Early tests of the disposal simulation indicated that the mounding only occurred when the percent moisture was lower than some value thus dispersion was anticipated to be different for material from the bottom of the hopper than for the top. A two-layer test in a 9 foot deep test tank was conducted as part of the large tank tests, to examine the effect of percent moisture gradient in the haul vessel on dispersion. This test used a percent moisture of 100 for the bottom half of the material and 400 for the upper half.

There was evidence from our earlier tests that mounding does not occur above some percent moisture. A set of tests were conducted in 28 gallon aquariums to establish the relationship between percent moisture and mounding of the material on the bottom. For the two materials of interest, tests were conducted using five different percent moisture mixtures at values near the liquid limits for the materials.

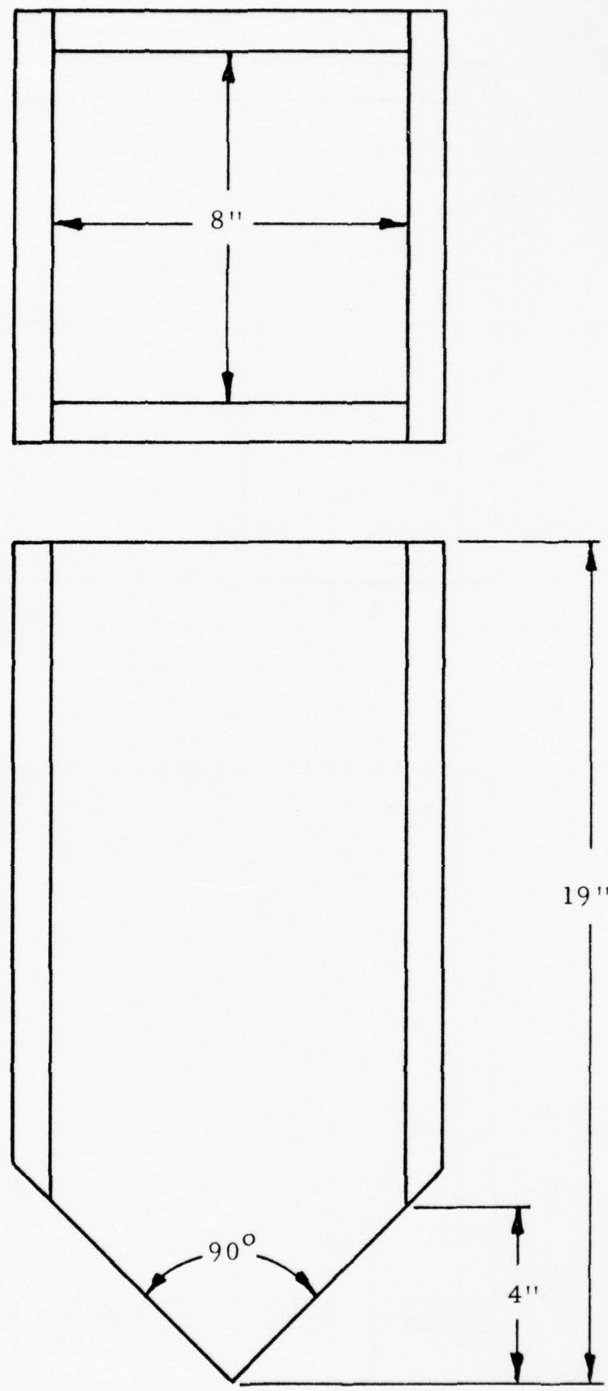



Figure 3-64. Simulated Hopper Dredge Dump Container

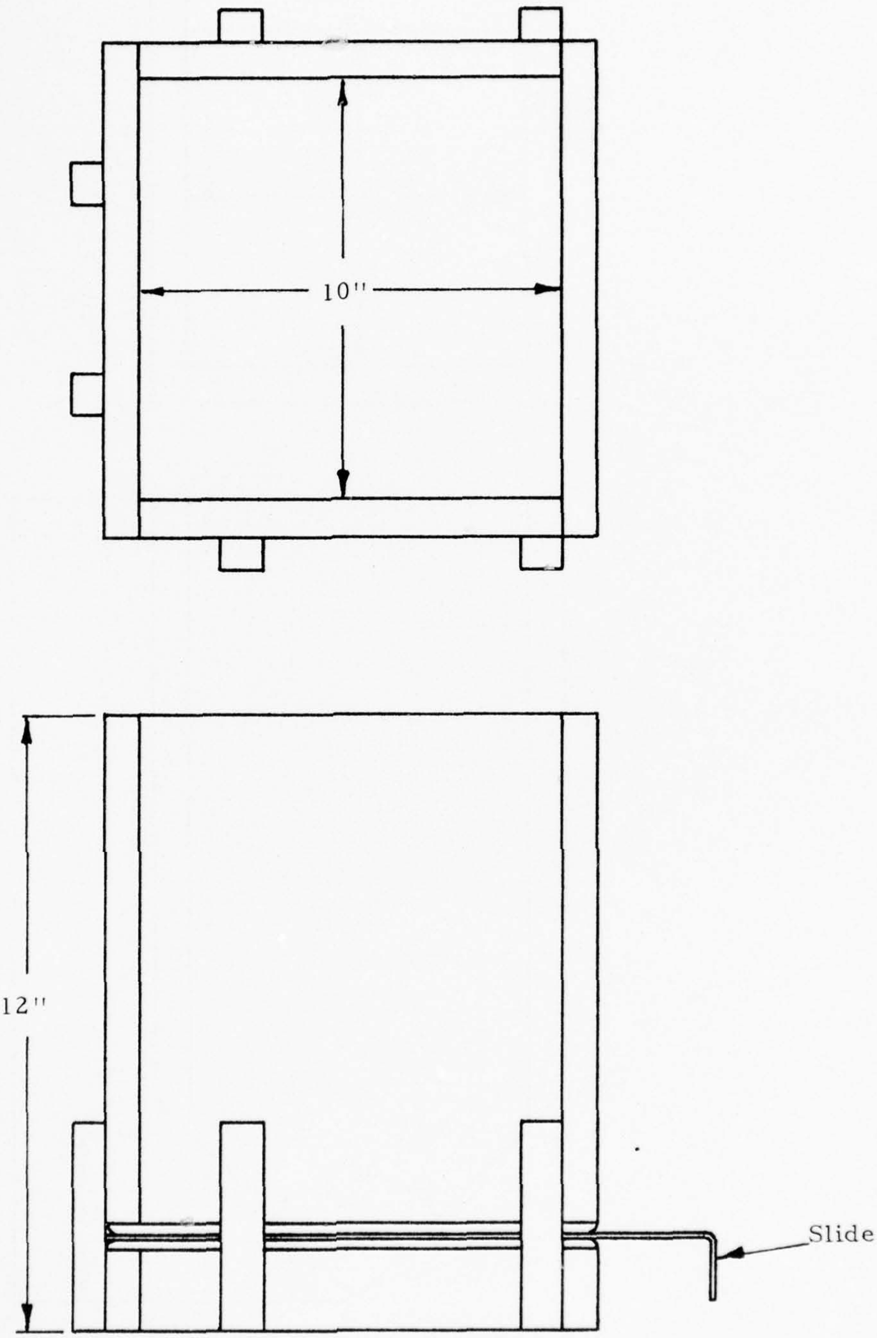



Figure 3-65. Simulated Split Bottom Barge Dump Container

The following sections describe the results obtained in the large tank simulations (descent, impact, and bottom flow) and the small aquariums (mounding as a function of percent moist...e)

b. Scaling Considerations

Before the simulation test data can be properly evaluated it is important to consider the basis for extrapolation of reduced scale observations to predictions of behavior in the full size. In general, physical phenomena can be modelled and predictions made from reduced scale observations, provided that all corresponding forces affecting the phenomena can be scaled at the same ratio. When this condition is met the modelling relationship is called a "dynamical similitude."

For dumping effects presently under study, the forces affecting the phenomena of interest are primarily inertia forces (cloud mass), gravity forces (cloud's negative buoyancy) and friction forces (skin friction of cloud and of individual particles). Unfortunately, it is impossible to design a practical modelling relationship where the ratios between all three of these forces will be constant (dynamical similitude). It can be shown that, where scaling is based on equivalent Froude numbers for both full and scaled conditions, there will be a constant ratio between gravity and inertia forces [21]. However, in order to preserve a constant ratio between frictional and inertia forces, it is necessary to scale on the basis of Reynolds number. These scaling ratios are defined as:

$$\text{Froude number} = \frac{V^2}{Lg}$$

$$\text{Reynolds number} = \frac{\rho V L}{\mu}$$

where $V$ is velocity

$L$ is length

$g$ is acceleration due to gravity

$\rho$ is density

and $\mu$ is viscosity

From these ratios it can be seen that scaling to achieve equivalent Froude number and, at the same time, equivalent Reynolds number, would require either scaling gravity as well as length and velocity, or conducting tests in a medium other than water in order to scale density or viscosity. These alternatives are clearly impractical.

There is, however, a reasonable basis for making some predictions through scaled models since, for most of the phenomena under study, it is probably reasonable to assume that frictional forces are relatively minor as compared to gravity and inertia forces. If this is the case a scaling based on Froude number will preserve sufficient dynamical similitude to make reasonable predictions of dump behavior.

Through a comparison of test results this has been shown to be the case in the descent phase of dumping for the range of materials under consideration (this will be discussed further in the following section). It also appears to be valid for much of the bottom flow phase. Thus, it appears valid to extrapolate many of the test results to full size predictions on the basis of Froude number. Although prediction of precise quantitative effects is not always possible from these test runs, generally, effects observed in the tests will correspond to full scale phenomena.

It should be pointed out, however, that dynamical similitude is no longer maintained when frictional forces become significant as compared to gravity and inertia forces. This may be the case,

for example, where a descending cloud is sufficiently slowed by entrainment during descent so that the cloud approaches neutral buoyancy; or, for example, when flow of material along the bottom is significantly slowed by loss of momentum.

Furthermore, it should be noted that in the reduced scale tests, the quantities that have been scaled are gross dump characteristics (size and volume) while the micro characteristics (particle size and density) have not been scaled. Thus, scaling relationships apply only when the dumped material acts as a cloud or slug and individual particle behavior is insignificant. Where the observed phenomena result from individual particle behavior, for example, in long term diffusion of dumped material, scaling relationships will be entirely different. When individual particle behavior is the predominant effect, for example, in settling of suspended material, scaling relationships no longer apply since the material is acting in the full scale (particle size and weight are unscaled).

c. Large Tank Simulations

In this section the effects observed in the large tanks, at depths of 2, 4, and 9 feet, will be presented. The first section presents a qualitative description of the phenomena observed during these tests. The remaining sections relate the dump characteristic variables that are under study to these phenomena and present quantitative evaluations of the observed effects.

(1) General Description

The material dumps observed in this phase of the program could be described as following three distinct phases:

descent phase
collapse phase
bottom flow phase

The descent phase begins with the release of the dumped material and extends to the point when the descending cloud first contacts the bottom. This phase is characterized by acceleration/deceleration of the descending cloud and by entrainment of ambient fluid into the cloud.

The collapse phase begins when the cloud contacts the bottom and ends when the major part of the cloud has lost its downward momentum. The collapse phase is characterized by a dissipation of the vertical momentum of cloud particles or a redirection of that momentum into a horizontal flow of the cloud material.

The bottom flow begins at the termination of the collapse phase. During this phase, the dumped material may, under certain conditions, flow as a fluid mud across the tank bottom. The characteristics of each of these typical dump phases will be discussed in greater detail below.

Descent Phase - Figure 3-66 shows the typical form of descent profiles observed in the large tank tests. This figure shows position of the cloud (depth below the surface) as a function of time since release. Typically, three stages can be observed in the descent phase. In the first (acceleration) the cloud starts from zero velocity and accelerates towards an equilibrium descent velocity.

As descent velocity increases, drag on the cloud, which is proportional to the velocity squared, also increases until the point where it approaches the negative buoyancy of the cloud. At that point the negative buoyancy force (downward) is nearly matched by the drag force (upward) and the cloud continues falling at a fixed velocity (equilibrium velocity). The cloud would continue falling at the equilibrium velocity until bottom impact if there was no entrainment.

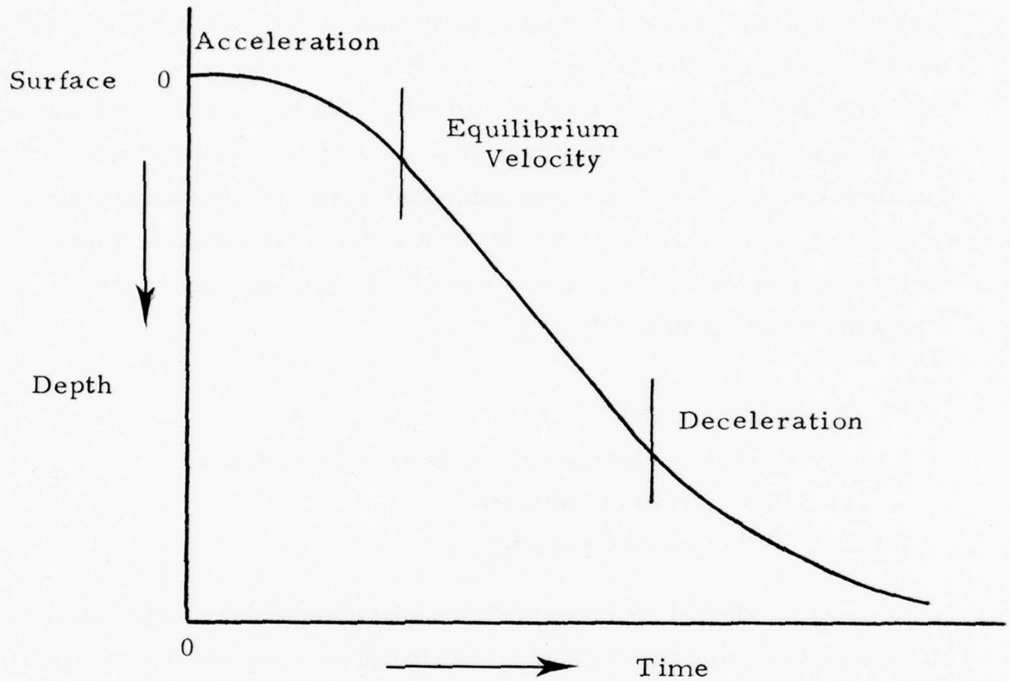



Figure 3-66. Typical Descent Profile for Dumped Material

During descent, however, due to relative motion in the ambient fluid, the cloud is entraining ambient fluid through its leading edge. This entrainment causes the cloud to grow so that the area of its leading edge expands, increasing the drag force on the cloud. Since the negative buoyancy remains constant, this increased drag causes a net decelerating force on the cloud during the final stage of its descent. In addition, conservation of momentum causes a reduction in descent velocity as the cloud expands by entrainment so that the final descent stage is characterized by a deceleration related to the entrainment rate. Entrainment rate is related to the descent velocity of the cloud and the face area of the cloud. An entrainment coefficient ( $\alpha$ ) is defined by the following relationship [19].

$$E = 2\pi b^2 \alpha V$$

where E is entrainment (volume per unit time)
b is cloud radius
V is cloud velocity

In general, $\alpha$ should be dependent on the properties of the material cloud and the ambient fluid and the turbulence structure within the cloud. For given dumping and material conditions, $\alpha$ should be essentially constant. The value of $\alpha$ has been estimated for the lab simulation dumps from the movie data by the following equation:

$$\alpha = \frac{E}{2\pi b^2 V}$$

where E = change in cloud volume per unit time

$$= \left( \frac{4}{3}\pi \left(b_2^2\ r_2 - b_1^2\ r_1\right) / (t_2 - t_1)\right)$$

$V$ = velocity of cloud = $(d_2 - d_1)/(t_2 - t_1)$

with b and r the horizontal and vertical cloud radii respectively

$d$ = the cloud depth

$t$ = time since start of the drop

and the subscripts refer to values at the start and end of a small time increment.

In most cases the value of $\alpha$ was found to be relatively constant during a single test dump, however, in some cases abrupt changes were noted in the entrainment coefficients during descent. These appeared to be essentially random effects related, perhaps, to changes in cloud shape and configuration.

It should be noted that although the descent profile described above was typical, there were variations from test to test. In general, the acceleration stage was short with the cloud attaining equilibrium velocity very shortly after release. In some cases, particularly with clay material of relatively high percent moisture, the initial stage was actually a deceleration with the material apparently reaching a velocity higher than equilibrium velocity even before it cleared the hopper. Once clear, the cloud experienced a drag that decelerated the material towards equilibrium velocity. In these cases a plot of the descent profile has a decreasingly negative slope in the first stage. For the case of low percent moisture silts, on the other hand, the initial acceleration was usually very high with the cloud reaching equilibrium velocity almost immediately on release from the hopper. In these cases plots of the descent profile had no clear acceleration phase and appeared to start out at equilibrium velocity.

In a number of cases the last descent stage (deceleration) was not

apparent and the cloud appeared to drop at equilibrium velocity until the bottom was encountered. In general, this occurred when the material dumped was of low percent moisture and entrainment was minimal. Another effect frequently noted was that in many cases the dump could not be characterized as a single cloud but instead descended as a series of overlapping or separate clouds.

Collapse Phase - The collapse phase is essentially a transition phase where the downward momentum of the descending cloud is arrested and partially redirected into a horizontal spreading of the cloud. This phase begins with the first contact of the cloud on the bottom and ends when the vertical momentum is essentially dissipated. It initiates the horizontal flow of waste material along the bottom.

During collapse vertical motion is suppressed by the bottom and some fraction of the momentum is lost as material is deposited on contact while the remaining momentum is redirected into horizontal and vertical flow of the more fluid material. Generally, the higher the percent moisture of the waste cloud the greater the fraction of momentum that is directed to horizontal flow while for low moisture content dumps most of the momentum is lost on impact. Some measure of this distribution of the collapse energy has been made by comparing the impact velocity to the initial velocity of the bottom flow generated by the impact.

Bottom Flow - Bottom flow is initiated by the collapse phase when the arrested vertical momentum of the waste cloud is redirected into a horizontal flow. When this occurs, a horizontal cloud may develop and spread rapidly along the bottom. The development of the horizontal cloud is also dependent upon the original concentration of the waste material and the entrainment during descent. A true fluid mud flow will occur only for sediment

concentrations greater than about 10 gm/l but less than about 170 gm/l [22].

A fluid mud flow can be driven by a) its own initial momentum, b) the density difference between the cloud and the ambient water, and c) gravity, when the bottom is not level. The resisting forces to the mud flow include drag forces due to the skin friction and form drag of the advancing cloud as well as frictional drag on the bottom. Where the drag forces are sufficiently large the deceleration of the advancing mud flow can be observed and measured from the lab simulation tests. Bottom flow velocity profiles showing the velocity of the face of the advancing horizontal cloud as a function of distance from the point of impact typically show an exponential deceleration.

(2) Discussion of Observed Effects

In this section the quantitative effects observed in the large tank test dumps will be presented. The dump size, depth of dump, and the percent moisture of the dump material appear to have the most significant effect on the disposal phase. The method of discharge (barge or hopper configuration) and the ambient water (salt water or fresh water) appear to have little or no influence on dump characteristics for the materials and test conditions utilized in this study.

Depth and size of dump are, in fact, variables that are related through scaling since a small dump volume dropped in shallow water shows similar characteristics to a larger volume dropped in a greater depth. Thus, once the scaling relationships are established, there is some equivalence in the effects observed from tests that vary dump volume while holding depth constant and those that vary depth of dump while holding volume constant.

The variable that appeared to have the most significant influence on the disposal phase was the moisture content of the dumped material (presented here as percent moisture). At moisture contents below a certain critical value the materials behaved essentially as one or more solid pieces falling to the bottom with little change of shape or dispersion of material. At moisture contents above another somewhat higher critical value, the materials behaved essentially as fluid clouds with constant entrainment characteristics. Between these critical values was a transition range where the dumped materials appeared to undergo changes in entrainment characteristics during descent. The transition range itself was a function of material (differs for clay and silt) and test results suggest that it is related to size and depth of dump in a fairly simple way.

In the following sections, the results obtained will be related to specific tests and Table 3-18 presents a summary of the characteristics of each test. Quantitative data were obtained from measurements made on still photographs at known times after release.

Effects of Discharge Method and Water Type

The effects of method of discharge (simulated barge or hopper) on dump characteristics were observed by comparison of tests 10 and 15 for clay dumps and tests 9 and 16 for silt dumps. In each case the material dropped was 10 liters in volume and was about 190 PCM (percent moisture). The drops were in four-foot water depth.

The descent profile and the cloud's horizontal diameter, as a function of time after release, are shown in Figure 3-67 for both discharge methods, with clay material. From this figure it can be seen that both the descent profile and the cloud diameter are similar in form and shape indicating little effect on descent characteristics by the release method used.

TABLE 3-18 - Summary of Large Tank Tests

| Test No. | Ambient Conditions: Water (Fresh/Salt) | Ambient Conditions: Water Depth (ft) | Ambient Conditions: Release (Hopper/Barge | Dump Material: Type (Clay/Silt | Dump Material: Volume (liters) | Dump Material: Moisture Content (PCM) | Descent Statistics: Average Descent Velocity (ft) | Descent Statistics: Average Entrain-ment Coefficient | Bottom Flow: Average Velocity 2' to 6' from impact (ft/sec) | Bottom Deposit: Average settling 2' to 6' from impact (mg/cm$^2$ dry) | Mound Depth: Average Inches | Mound Depth: Max. Inches |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | F | 4 | bag | C | 10 | 131 | 2.3 | 0.25 | --- | --- | --- | 4 |
| 2 | F | 4 | B | C | 10 | 135 | 1.5 | ---- | .13 | 10 | 3.5 | 4.5 |
| 3 | F | 4 | H | C | 10 | 121 | 2.0 | ---- | .06 | 10 | 3.0 | 4.0 |
| 4 | F | 4 | H | C | 10 | 183 | 1.7 | 0.40 | .40 | 60 | neg. | .5 |
| 5 | F | 4 | H | C | 10 | 535 | 1.3 | 0.5 | .27 | 83 | neg. | |
| 6 | F | 4 | H | S | 10 | 100 | 1.5 | 0.15 | .10 | 11 | 3.0 | 5.0 |
| 7 | F | 2 | H | S | 10 | 207 | 5.0 | 0.0 | .40 | 460 | neg. | |
| 8 | F | 2 | H | C | 10 | 200 | 4.4 | 0.0 | .24 | 376 | neg. | |
| 9 | F | 4 | N | S | 10 | 190 | 1.8 | 0.30 | .33 | 620 | neg. | .25 |
| 10 | F | 4 | H | C | 10 | 190 | 1.7 | 0.25 | .36 | 494 | neg. | .5 |
| 11 | F | 4 | H | S | 10 | 67 | 2.3 | 0.0 | .20 | 73 | 3" | 6" |
| 12 | F | 4 | H | C | 10 | 190 | 1.4 | 0.25 | .19 | 179 | neg. | |
| 13 | F | 4 | H | S | 10 | 403 | 1.4 | 0.25 | .33 | 322 | neg. | |
| 14 | F | 4 | H | C | 1 | 191 | .8 | 0.30 | .03 | 9 | neg. | |
| 15 | F | 4 | B | C | 10 | 191 | 1.5 | 0.30 | .45 | 381 | neg. | .3 |
| 16 | F | 4 | B | S | 10 | 191 | --- | ---- | .33 | 467 | neg. | |
| 17 | F | 4 | H | S | 1 | 187 | .7 | 0.25 | .10 | 59 | neg. | |
| 18 | F | 4 | H | S | 5 | 187 | 1.0 | 0.30 | .31 | 409 | neg. | |
| 19 | S | 4 | H | S | 10 | 120 | 3.9 | 0.0 | .09 | 62 | 4.5 | 6.0 |
| 20 | S | 4 | H | C | 10 | 95 | 2.3 | 0.0 | .08 | 54 | 4.0 | 6.0 |
| 21 | S | 4 | H | S | 10 | 410 | 1.2 | 0.35 | .19 | 332 | neg. | |
| 22 | S | 4 | H | C | 10 | 422 | 1.5 | 0.6 | .25 | 278 | neg. | |
| 23 | F | | | | | run cancelled - see #24 | | | | | | |
| 24 | F | 9 | H | S | 10 | 207 | 1.2 | 0.3 | not taken for deep drops | | neg. | |
| 25 | F | 9 | H | C | 10 | 190 | 1.3 | 0.3 | | | neg. | |
| 26 | F | 9 | H | C | 10 | part 112 part 385 | 2.2 | 0.25 | | | 3.0 | 5.0 |
| 27 | F | 9 | N | S | 10 | 84 | 3.1 | 0.2 | | | 2.0 | 4.0 |
| 28 | F | 9 | H | S | 10 | 112 | 3.3 | 0.3 | | | 3.0 | 7.25 |

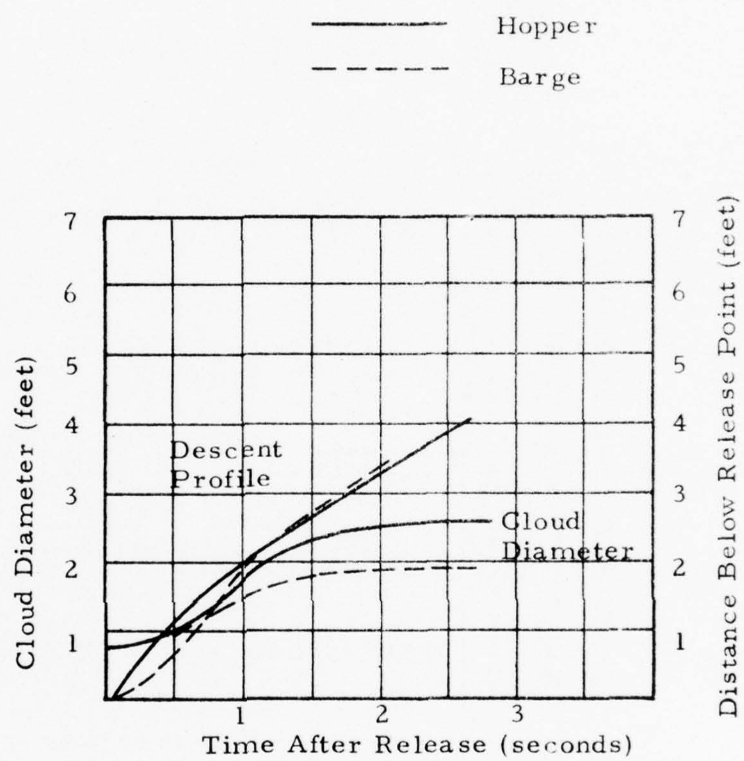



Figure 3-67. Effect of Method of Release - Clay

The descent profile shows almost identical descent velocities at all depths except immediately after discharge where the barge release appeared to experience a slightly slower initial acceleration. This initial velocity difference is probably caused by the difference in material elevations above the water surface prior to release. The cloud diameter profiles appear very similar in shape but differ by as much as about 20 percent at some points in descent - the discharge from the barge producing a slightly smaller cloud.

This suggests that the almost instantaneous dump produced by the simulated hopper creates a higher velocity early in the descent, with increased entrainment,thus a larger cloud diameter. Once the descent velocities become the same (parallel lines in the descent profile) the cloud diameters appear to behave the same. Thus the difference in descent characteristics seems to be generated during the period in the descent at which the descent velocities are different and this is most probably a small effect.

For the case of silt material tests 9 and 16 were available for comparison of effects of discharge method, but due to a camera malfunction during test 16, data on descent and cloud diameter profiles could not be generated. On the basis of information shown in the Table 3-19, however, there appears to be little significant difference in dumping characteristics by discharge method for silt or clay. It should be noted that for both silt and clay there are no appreciable differences by discharge method in average descent velocity, entrainment coefficient or average bottom flow velocity.

For evaluation of effects of the type of water (fresh or salt) in which the dump occurred, the following simulation dumping tests were available:

TABLE 3-19 - Effect of Release Method (barge versus hopper)

| | Clay | | Silt | |
|---|---|---|---|---|
| Test No. | 10 | 15 | 9 | 16 |
| Release method | hopper | barge | hopper | barge |
| Average descent velocity (ft/sec) | 1.7 | 1.5 | 1.8 | 1.5-2.0 |
| Average entrainment coefficient | 0.25 | 0.30 | 0.30 | ------- |
| Average bottom flow rate (ft/sec) (over first 6 feet) | 0.36 | 0.45 | 0.33 | 0.33 |

tests 3 and 20 for clay of low moisture content (about 100 PCM)
tests 5 and 22 for clay of high moisture content (about 500 PCM)
tests 11 and 19 for silt of low moisture content (about 100 PCM)
tests 13 and 21 for silt of high moisture content (about 400 PCM)

In each case the material dropped was 10 liters in volume and it was dropped in 4-foot water depth.

In Figures 3-68, 3-69, and 3-70, the descent profile and the cloud's horizontal diameter are plotted as a function of "time since release" for each of these comparisons except the case of low moisture content clay. In that case a camera malfunction caused loss of data so that the descent profile for test 3 could not be generated. Table 3-20 presents the relevant dump statistics for all four comparisons.

From these figures it can be seen that for high percent moisture in both clay and silt, the descent profile and cloud size profile do not differ significantly between salt and fresh water (Figures 3-68 and 3-70). For the clay dump the salt water drop shows a somewhat higher average descent than the fresh water drop but this may be due to measurement and test variability and without further replication of tests the difference cannot be considered significant.

For the low moisture content comparison (Figure 3-69) it is interesting to note that, at least for silt, the descent profile is considerably steeper in salt water than in fresh. The average descent velocity calculated for runs 3 and 20 (see Table 3-20) indicate that the same is not true, however, for clay dropped in fresh and salt water. Again there is insufficient replication to verify the trend here but the difference appears greater than measurement and test variations, suggesting that for the low PCM silt material used in these tests descent velocities are higher in

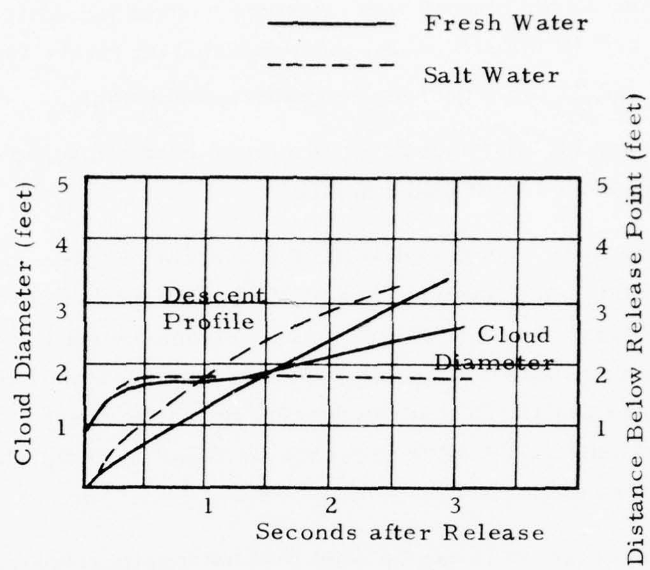



Figure 3-68. Effect of Water Type (fresh or salt) - Clay Approximately 500 PCM (Tests 5 and 22).

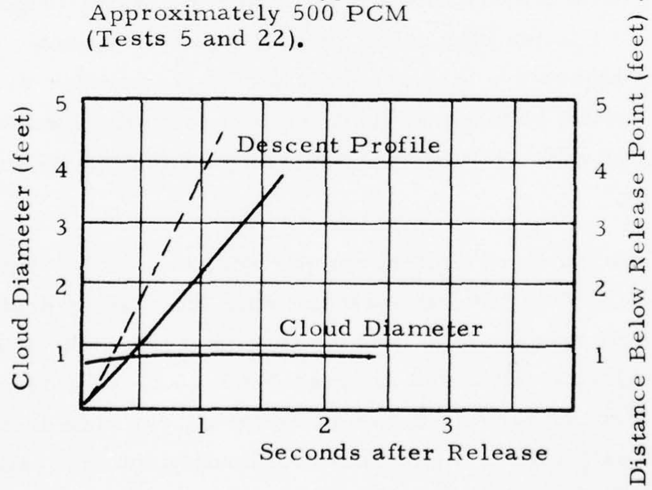



Figure 3-69. Effect of Water Type (fresh or salt) - Silt Approximately 100 PCM (Tests 11 and 19)

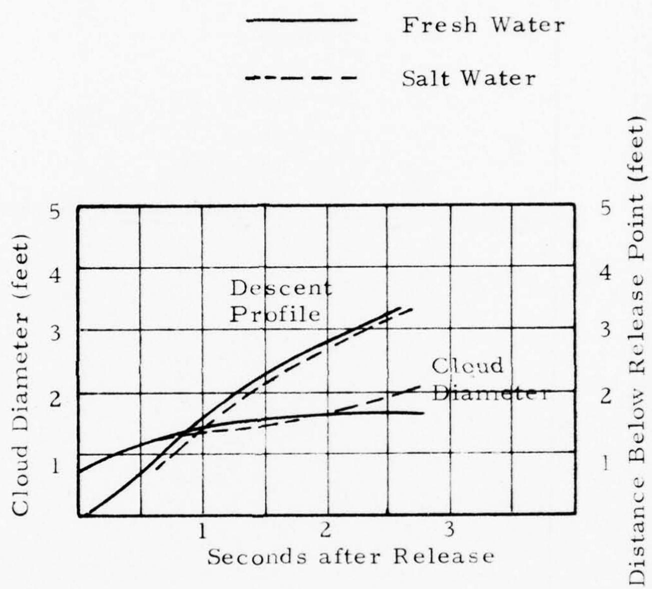



Figure 3-70. Effect of Water Type (fresh or salt ) Silt, Approximately 400 PCM (Tests 13 and 21)

TABLE 3-20 - Effect of Water-Type (Fresh or Salt) on Disposal Phase

| | Clay | | | | Silt | | | |
|---|---|---|---|---|---|---|---|---|
| Test No. | 3 | 20 | 5 | 22 | 11 | 19 | 13 | 21 |
| Water (Fresh or Salt) | F | S | F | S | F | S | F | S |
| Percent moisture | 121 | 95 | 535 | 422 | 67 | 120 | 403 | 410 |
| Average descent velocity (ft/sec) | 2.0 | 2.3 | 1.3 | 1.5 | 2.3 | 3.9 | 1.4 | 1.2 |
| Average Entrainment Coefficient | ---- | 0 | .25 | .30 | 0 | 0 | 0.25 | 0.35 |
| Average bottom flow rate (ft/sec) over first 6 feet | 0.06 | 0.08 | 0.20 | 0.29 | 0.20 | 0.09 | 0.33 | 0.19 |

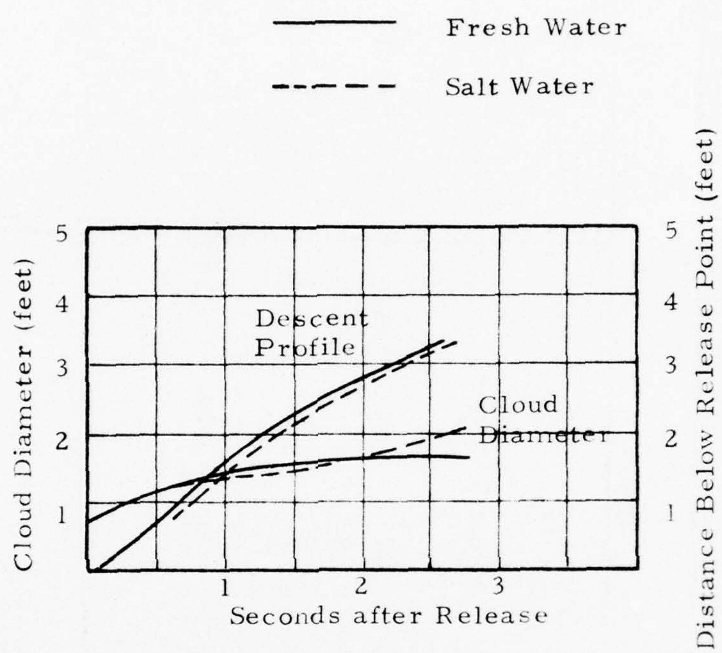



Figure 3-70. Effect of Water Type (fresh or salt ) Silt, Approximately 400 PCM (Tests 13 and 21)

TABLE 3-20 - Effect of Water-Type (Fresh or Salt) on Disposal Phase

| | Clay | | | | Silt | | | |
|---|---|---|---|---|---|---|---|---|
| Test No. | 3 | 20 | 5 | 22 | 11 | 19 | 13 | 21 |
| Water (Fresh or Salt) | F | S | F | S | F | S | F | S |
| Percent moisture | 121 | 95 | 535 | 422 | 67 | 120 | 403 | 410 |
| Average descent velocity (ft/sec) | 2.0 | 2.3 | 1.3 | 1.5 | 2.3 | 3.9 | 1.4 | 1.2 |
| Average Entrainment Coefficient | ---- | 0 | .25 | .30 | 0 | 0 | 0.25 | 0.35 |
| Average bottom flow rate (ft/sec) over first 6 feet | 0.06 | 0.08 | 0.20 | 0.29 | 0.20 | 0.09 | 0.33 | 0.19 |

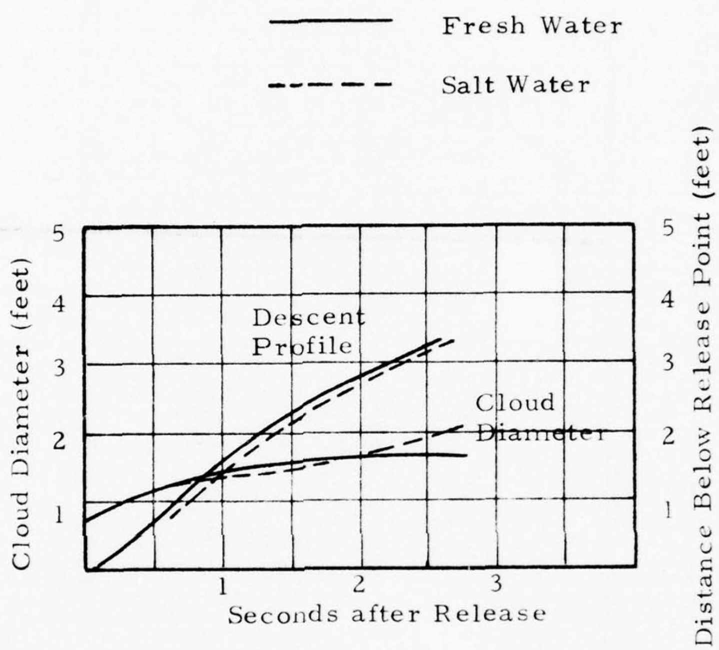



Figure 3-70. Effect of Water Type (fresh or salt ) Silt, Approximately 400 PCM (Tests 13 and 21)

TABLE 3-20 - Effect of Water-Type (Fresh or Salt) on Disposal Phase

| | Clay | | | | Silt | | | |
|---|---|---|---|---|---|---|---|---|
| Test No. | 3 | 20 | 5 | 22 | 11 | 19 | 13 | 21 |
| Water (Fresh or Salt) | F | S | F | S | F | S | F | S |
| Percent moisture | 121 | 95 | 535 | 422 | 67 | 120 | 403 | 410 |
| Average descent velocity (ft/sec) | 2.0 | 2.3 | 1.3 | 1.5 | 2.3 | 3.9 | 1.4 | 1.2 |
| Average Entrainment Coefficient | ---- | 0 | .25 | .30 | 0 | 0 | 0.25 | 0.35 |
| Average bottom flow rate (ft/sec) over first 6 feet | 0.06 | 0.08 | 0.20 | 0.29 | 0.20 | 0.09 | 0.33 | 0.19 |

salt water. This may be due to a greater cohesiveness of the material in a salt water environment thus reducing form drag of the descending volume by holding its shape and compactness. In support of this hypothesis is the observation that the cloud diameter remains constant during descent and entrainment is negligible. Furthermore, the bottom deposit from the fresh water drop was in 5 distinct mounds while that of the salt water drop was a single mound indicating that less break up of the material(and hence less increasein form and frictional drag) occurred in the salt water drop.

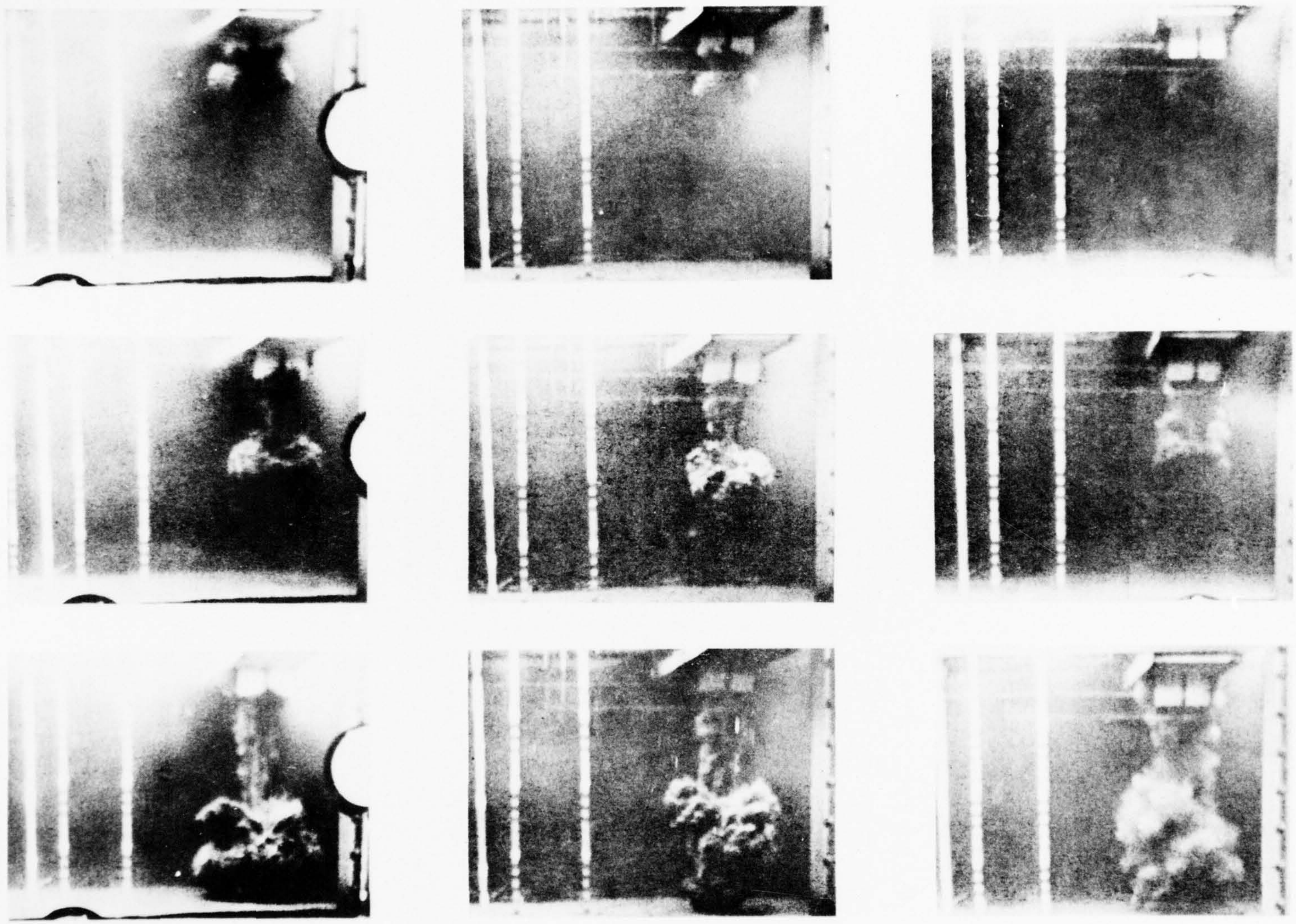
Without further replication it is not obvious whether the greater breakup and hence slower descent in the fresh water dump was a chance occurence or a real difference. It suggests the possibility that relatively solid material is better able to maintain its form and shape during descent in salt water.

In summary, the test results show little evidence of an effect on dumping characteristics due to the method of release (barge or hopper). The results also show little effect due to the ambient water (salt or fresh) except possibly a greater cohesiveness for low moisture content silt drops in salt water than in fresh water. Since the differences observed in these tests are not large, data can be combined from these tests for examination of the more pronounced effects of moisture content on the dump.

Effect of Size and Depth of Dump

The effect of dump size was investigated by comparison of tests 10, 12 and 14 for clay dumps and tests 9, 18 and 17 for silt dumps. In each of these drops the water depth was 4 feet and the moisture content of the material was about 90 percent moisture. The dump volume was varied from one to ten litres. Figures 3-71 and 3-72 show the descent phase for clay and silt drops for 3 different size dumps.

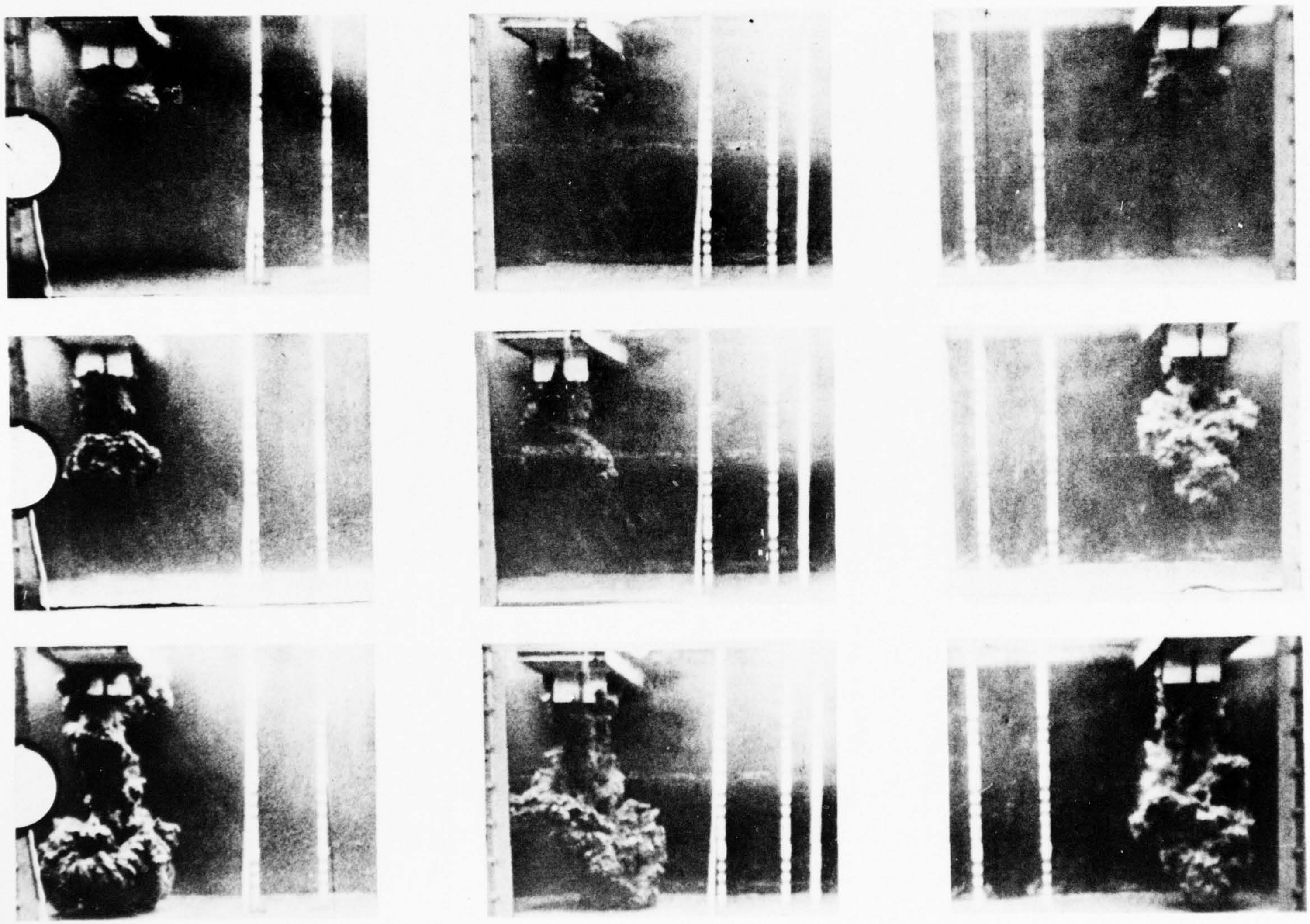
Test 10 Test 12 Test 14



10 liters 5 liters 1 liter

Figure 3-71. Effect of Dump Size on Descent Phase for Clay Material

Test 9 | Test 18 | Test 17



10 liters | 5 liters | 1 liter

Figure 3-72. Effect of Dump Size on Descent Phase for Silt Material

Figures 3-73 and 3-74 show the observed descent profile for both the clay and silt dumps as a function of the size of dump. Descent velocities increase and the effects of entrainment apparently decrease with increasing size of dump.

The large volume dump, in fact, falls at nearly an equilibrium velocity (indicated by a straight line profile) for most of its descent phase. The reason for this effect is that the cloud's surface area to mass ratio is far greater for the small dumps than it is for the large (since material density and PCM are held constant in all runs). While the cloud negative buoyancy is a mass related effect, the drag and entrainment phenomena are both area related effects. Thus, the relationships between weight effects and drag or entrainment effects will alter with dump size, weight being roughly in proportion to size cubed and drag or entrainment to size squared so that drag is greater in proportion to negative buoyancy for smaller dumps.

There is some indication from the descent profiles that the clay dumps experience a greater initial acceleration than the silt dumps with the clay perhaps accelerating while in the hopper so as to enter the water at a velocity higher than its equilibrium velocity. This may be due to a greater tendency for silt to cling to the hopper sides on release and the effect may be a peculiarity of the scale and design of the test hopper. In any event these characteristics of the release do not appear to influence the ultimate descent or disposal pattern of the material.

Table 3-21 shows relevant dump statistics for both clay and silt, by size of dump. The average bottom flow velocity is greater with greater dump size due to the higher impact velocity of the larger dump. The amount of settled material ($mg/cm^2$) in the vicinity of impact is also greater with larger dump size reflecting both the amount of material released and the greater dispersion resulting from the smaller, slower descending dumps.

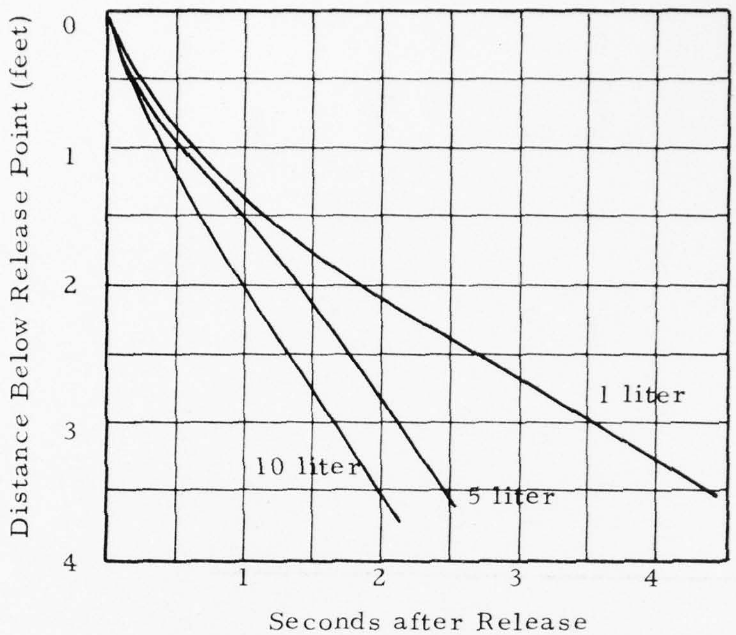



Figure 3-73. Descent Profile (leading edge of cloud) for Several Dump Sizes - Clay

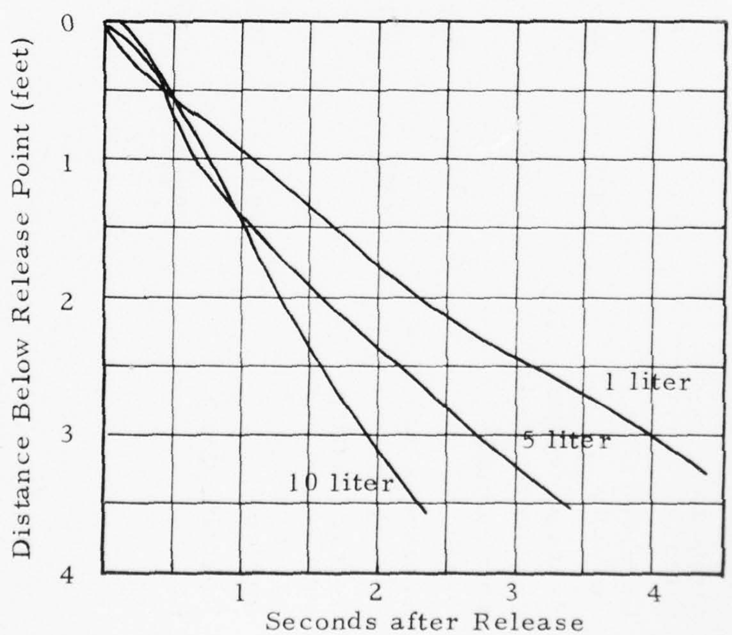



Figure 3-74. Descent Profile (leading edge of cloud) for Several Dump Sizes - Silt

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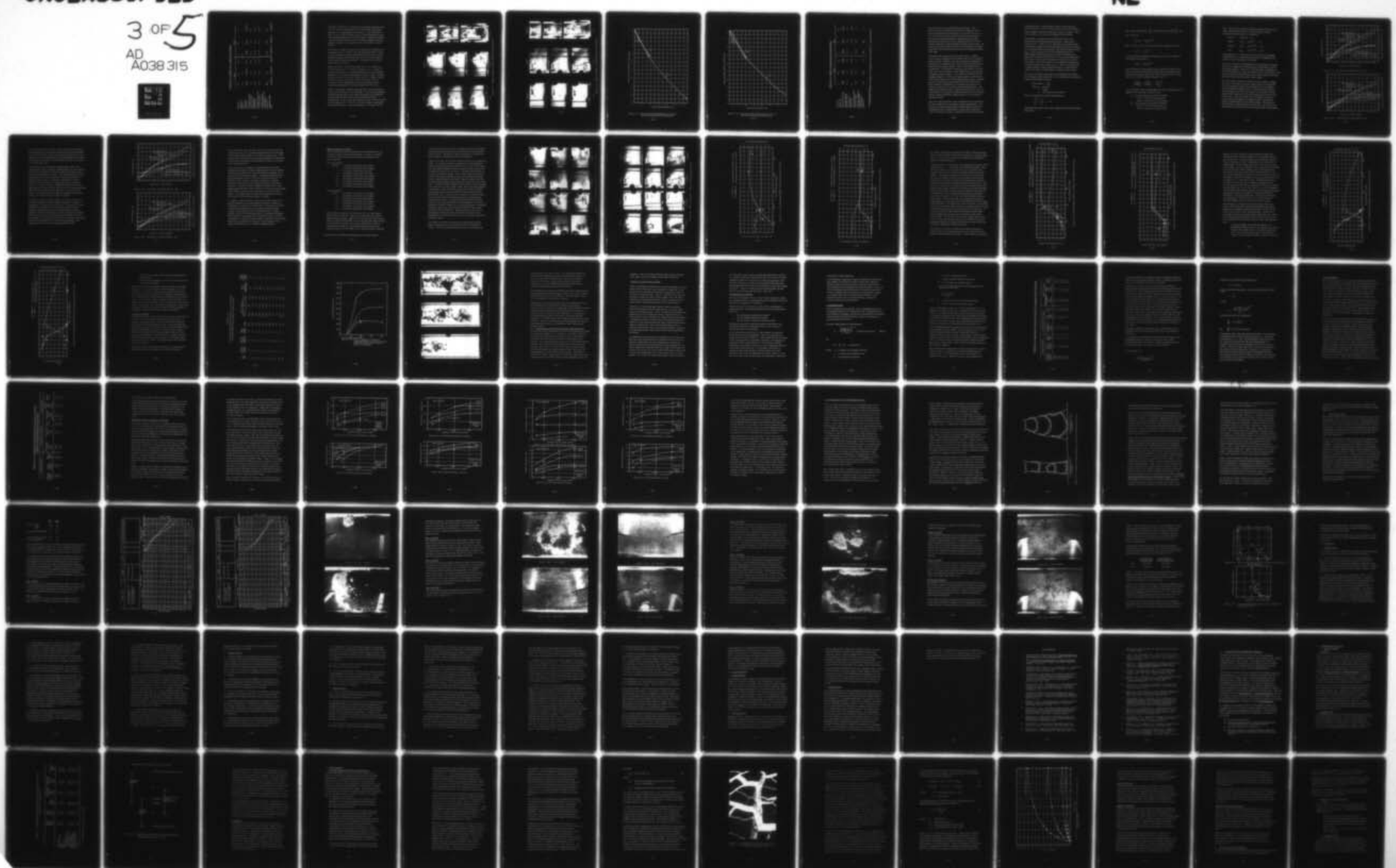

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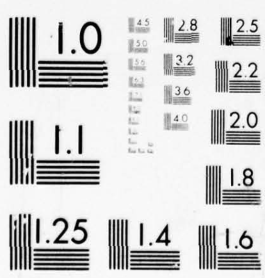


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TABLE 3-21 - Effects Due to Dump Size Variation

| | Clay | | | Silt | | |
|---|---|---|---|---|---|---|
| Test No. | 10 | 12 | 14 | 9 | 18 | 17 |
| Dump size (liters) | 10 | 5 | 1 | 10 | 5 | 1 |
| Observed Vorticity | high | medium | low | high | high | medium |
| Average descent velocity (ft/sec) | 1.7 | 1.4 | 0.8 | 1.8 | 1.0 | 0.7 |
| Entrainment coefficient | 0.25 | 0.25 | 0.30 | 0.30 | 0.25 | 0.30 |
| Average bottom flow velocity (ft/sec) | 0.30 | 0.19 | 0.03 | 0.33 | 0.31 | 0.10 |
| Average setting out of cloud in first 6 feet of flow ($mg/cm^2$) | 494 | 179 | 9 | 620 | 409 | 59 |
| Mounding depth (in) | negligible | negligible | negligible | negligible | negligible | negligible |

It is also of interest to note that, for the percent moisture used in these series of tests (about 190), entrainment coefficients appears to be independent of the dump size. This was true in spite of the fact that vorticity generated in the descending cloud was considerably greater for larger dumps. Although a greater vorticity did hold the cloud into a well defined shape, it did not appear to influence the entrainment rate nor the rate of cloud growth.

The effects of depth of dump were investigated by comparison of simulation tests 8, 10 and 25 for clay dumps and tests 7, 9 and 24 for silt dumps, as shown in Figures 3-75 and 3-76. In each of these drops the dump volume was 10 litres and the material was of about 190 percent moisture. The depth of the drop was varied from 2 to 9 feet.

Figures 3-77 and 3-78 show the observed descent profiles for clay and silt dumps as a function of depth of drop. Perhaps of greatest interest here is the observation that the descent profile appears to be essentially independent of depth of dump. Descent through the initial 4 feet of the 9 foot drop, for example, follows the same profile as that of the entire 4 foot drop. This is expected since the hydrodynamic effects during descent are essentially independent of the water depth below the cloud.

Table 3-22 presents relevant dump statistics, for both clay and silt for the test dumps at several depths. The distinction between the early descent stage, where the cloud falls at an equilibrium velocity, and the later stage, where entrainment effects predominate, is clear from these data as well as from the descent photos in Figures 3-75 and 3-76. The early stage, represented by the two foot dump, is characterized by a very high descent

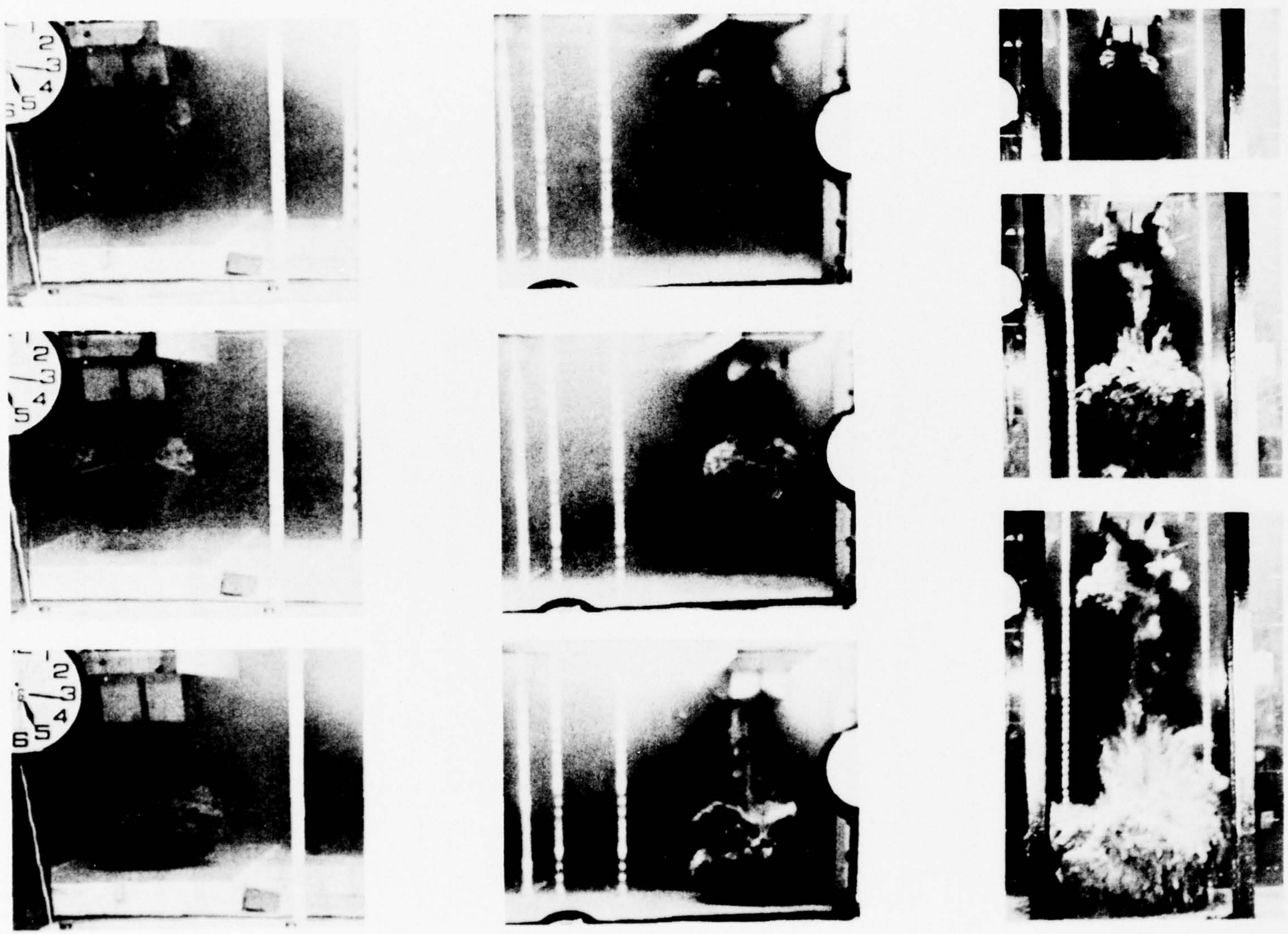
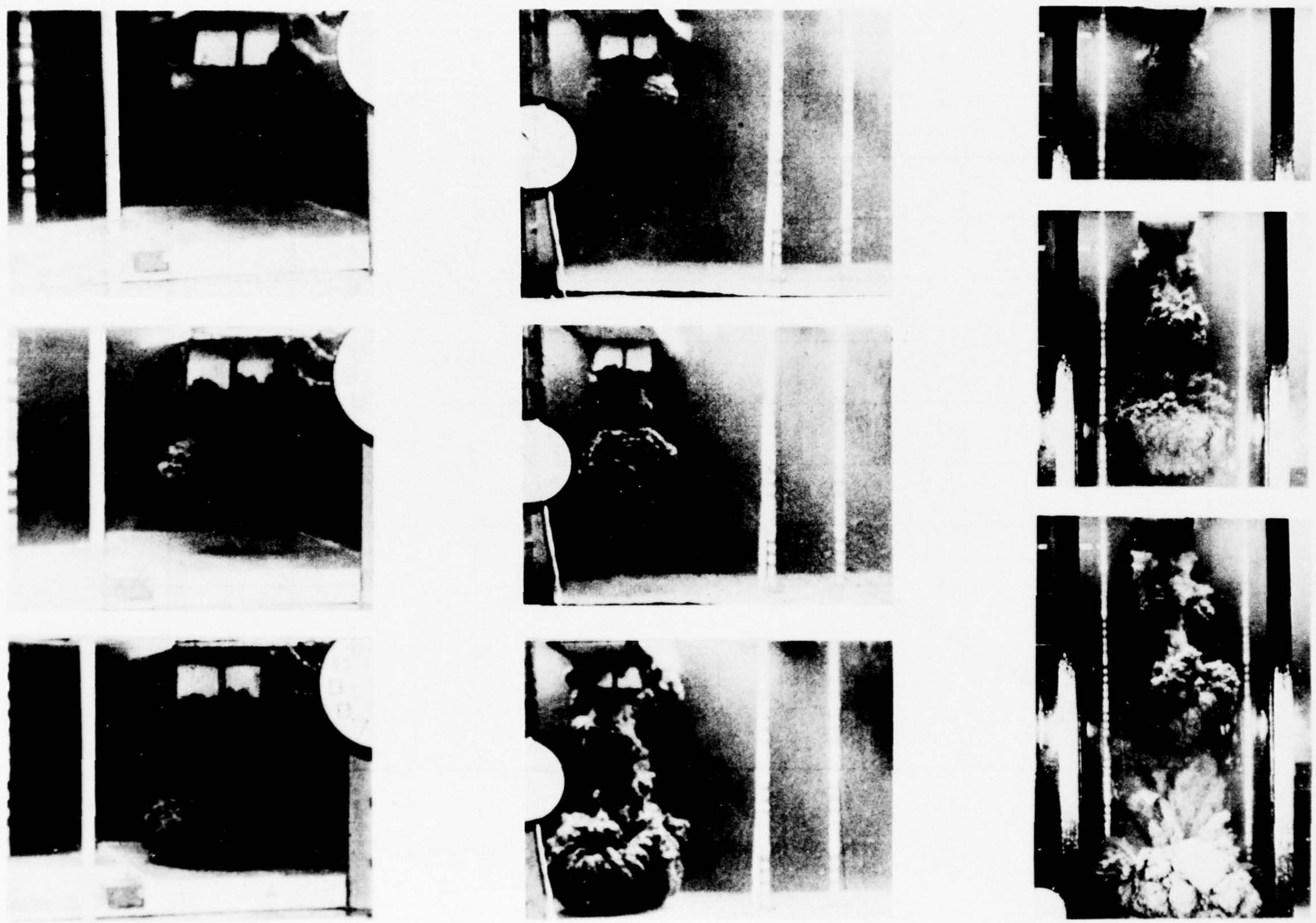



Figure 3-75. Effect of Depth on Descent Phase for Clay Material

Test 7 | Test 9 | Test 24



2 ft | 4 ft | 9 ft

Figure 3-76. Effect of Depth on Descent Phase for Silt Material

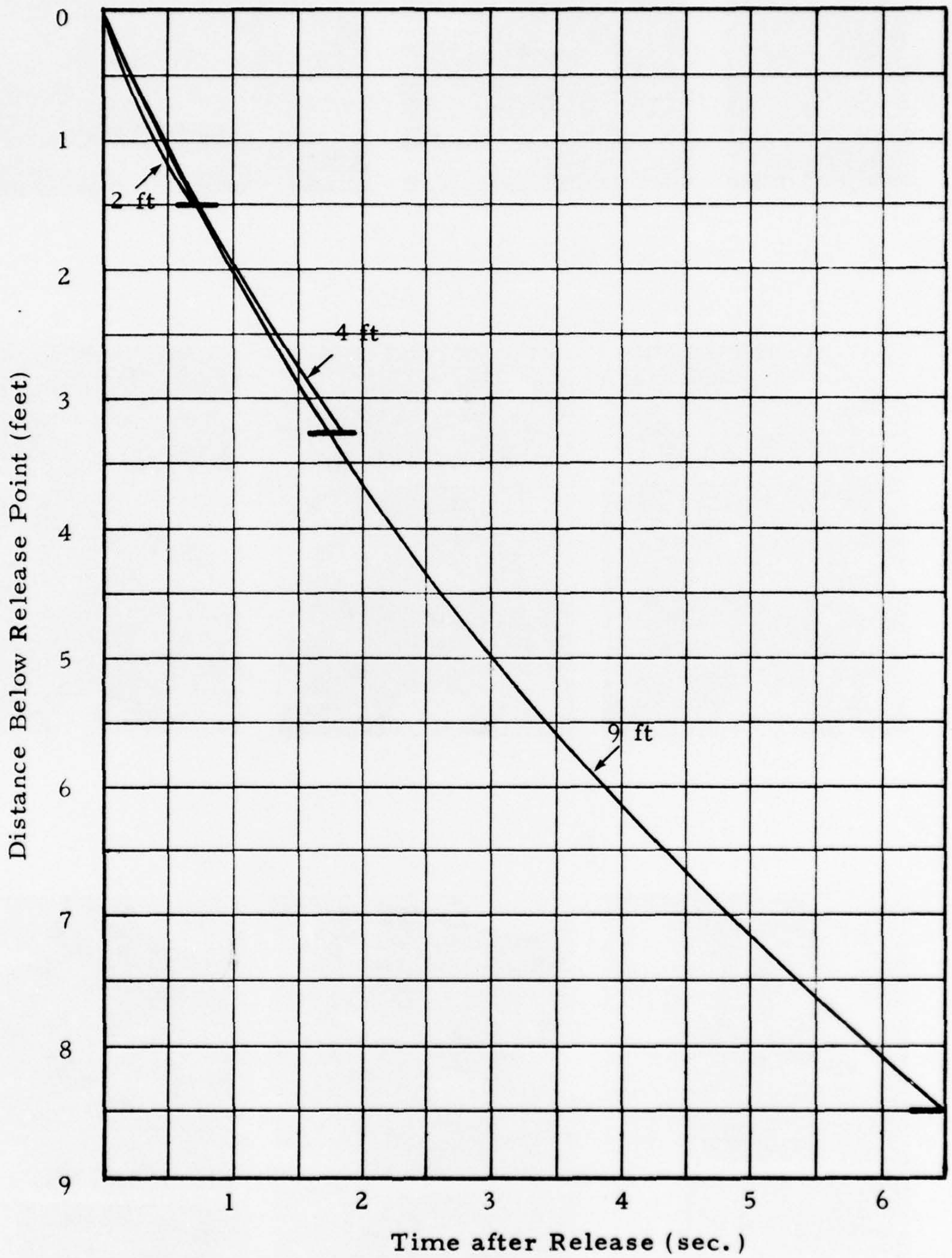



Figure 3-77. Descent Profile (leading edge of cloud) Vs Time after Release for Clay Material

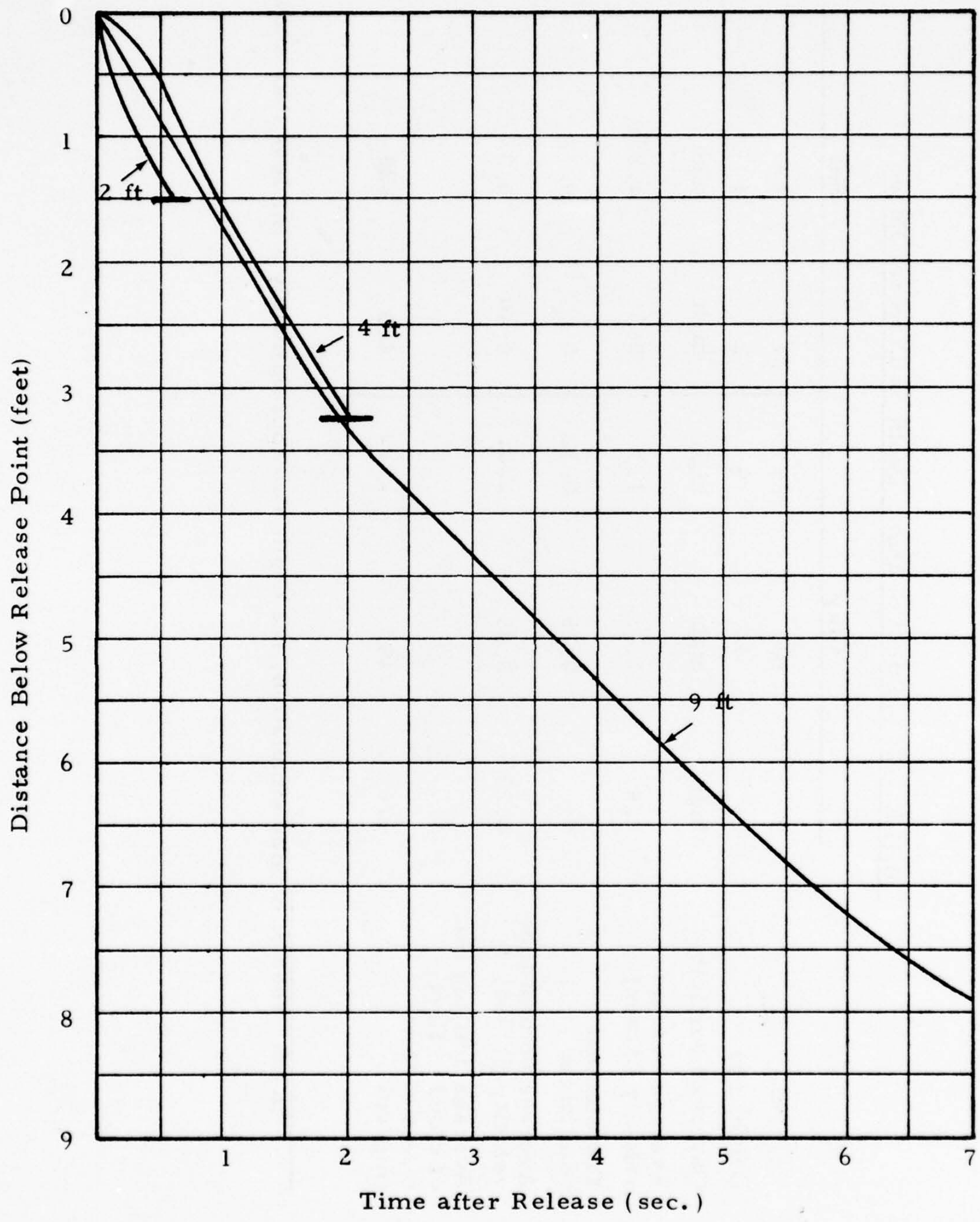



Figure 3-78. Descent Profile (leading edge of cloud) Vs Time after Release for Silt Material

TABLE 3-22 - Effect of Dump Depth on Disposal Phase

| | Clay | | | Silt | | |
|---|---|---|---|---|---|---|
| Test No. | 8 | 10 | 25 | 7 | 9 | 24 |
| Depth* (ft) | 2 | 4 | 9 | 2 | 4 | 9 |
| Observed vorticity | high | high | high | high | high | high |
| Average descent velocity (ft/sec) | 2.4 | 1.7 | 1.3 | 5.0 | 1.8 | 1.2 |
| Entrainment coefficient | 0 | 0.25 | 0.30 | 0 | 0.30 | 0.30 |
| Average bottom flow velocity (ft/sec) | 0.23 | 0.36 | ---- | 0.40 | 0.33 | ---- |
| Average settling out of cloud in first 6 feet of flow ($mg/cm^2$) | 376 | 494 | ---- | 460 | 620 | ---- |

*in the cases of hopper simulation, the depth was approximately 6 in. less than shown

velocity and a near zero entrainment coefficient. The descent velocity remains nearly constant as indicated by the nearly straight line descent profile over the first two feet. In the case of the 4 and 9 foot drops this early descent stage is clearly passed and entrainment is influencing the descent. Entrainment coefficients become significant (about 0.25) and remain essentially constant beyond the initial descent stage. At the same time the average velocity is decreasing for the deeper drops due to the increased deceleration in the expanded entrainment stage.

The bottom flow velocity shows no clear trend. A greater velocity may be expected for the shallow drop due to the higher impact velocity. However, since the cloud has not expanded by entrainment in a shallow drop, much of this momentum may be absorbed by the bottom impact. There is insufficient data to evaluate this. The deposit of material on the bottom, however, shows clear indication that, in the shallow drop, more material is deposited nearer impact. Less material is collected in the sampling range (2 to 6 feet from impact point) for the shallow drop but the distribution of this settled material shows a steeper increase towards the impact point than for the deeper drop. More material is deposited near the impact point and less is spread out beyond a 2 foot radius from the impact point. This suggests the possibility that, when entrainment is low, there is less tendency for material to disperse since most of the descent energy is then absorbed by impact and thus is not available to spread the cloud.

One of the major interests in the comparison tests of dump size and water depth was to investigate scaling relationships. It is possible to do this with the dump size and depth test series in combination since a small dump in a given water depth (say 1 liter at 4 feet) is, in fact, a scaled down version of the larger

combination since a small dump in a given water depth (say 1 liter at 4 feet) is, in fact, a scaled down version of the larger dump dropped at a greater depth (say 10 liters at 9 feet). These runs can therefore be used to investigate scaling effects.

The problem of scaling was discussed earlier and it was hypothesized that descent phenomena may be scaled about a Froude number. That is to say, that full size and small scale tests may be compared when both have the same Froude number. The reasoning behind this hypothesis was that the effect of inertia (e.g., in form drag) was the major influencing factor in determining descent and bottom flow behavior (though not in dispersion) and that frictional forces were of lesser importance. This is due to the relatively high mass to surface ratio of the dump cloud. The Froude number is the significant scaling ratio for effects dependent on inertia forces. Thus, the trajectory of a relatively heavy volume falling through air or water may be scaled on the basis of maintaining a constant Froude number.

The test series that varied dump size and depth proved useful in verifying the scaling hpothesis. For this purpose the following scaling relationships were used

$$\text{Froude number} = \frac{V^2}{Lg}$$

where V is velocity
L length
g acceleration due to gravity

Since g is constant for full and scaled effects,

$$\frac{V_f^2}{L_f} = \frac{V_s^2}{L_s} \qquad (1)$$

where the subscripts f and s refer respectively to full and scaled dimensions

The velocity scaling ratio, $\frac{V_f}{V_s}$, and length scaling ratio, $\frac{L_f}{L_s}$, are thus related by:

$$(V_f/V_s) = (L_f/L_s)^{1/2}$$

Since velocity is the ratio of length to time, equation (1) can be:

$$L_f/t_f^2 = L_s/t_s^2$$

where t is time, and thus the scaling ratio for time is related to the length scaling ratio by:

$$(t_f/t_s) = (L_f/L_s)^{1/2}$$

since volume (for similar geometry) is scaled by a length cubed and since density has been held constant by using similar dump materials in full and scaled tests, both volume and mass scaling ratios are related to length scaling factors by:

$$\left(\frac{Vol_f}{Vol_s}\right) = \left(\frac{M_f}{M_s}\right) = \left(\frac{L_f}{L_s}\right)^3$$

For verification of the scaling hypotheses these scaling ratios were applied to the following simulation tests:

a) for clay dumps (material at 190 PCM)
   - test 14 - 1 litre in 4 feet of water
   - test 25 - 10 litres in 9 feet of water

b) for silt dumps (material at 190 PCM)
   - test 17 - 1 litre in 4 feet of water
   - test 24 - 10 litres in 9 feet of water

The 1 litre tests were taken as scaled versions of the 10 litres tests. On the basis of the 1 to 10 litre volume ratio the following scaling factors were calculated:

volume: $Vol_f/Vol_s = 10$

length: $L_f/L_s = (10)^{1/3} = 2.15$

velocity: $V_f/V_s = (2.15)^{1/2} = 1.47$

time: $t_f/t_s = (2.15)^{1/2} = 1.47$

The length ratio of 4 feet to 9 feet is quite close to the length scaling factor of 2.15 and hence the tests are comparable over most of the descent depth for each case.

In order to verify the scaling hypothesis the appropriate scaling factors were applied to the depth ($L_f/L_s$) and time($t_f/t_s$) axes of the descent profile. Figures 3-79 and 3-80 show a comparison between the scaled up 1 litre tests and the "full" scale 10 litre tests. In each case the descent profile and the cloud diameter have been plotted as functions of time after dump release.

For both clay and silt these profiles are quite similar in shape and form. The differences, particularly in the descent profile are well within the normal measurement and test variability observed during simulation runs. It should be noted, however, that the hopper from which the dumps were released were not geometrically scaled and were in fact identical for both full and scaled runs. Thus, the geometries of the initial volumes as released from the hopper were different with a much flatter, pancake shaped volume for the smaller dump. The effects of this difference (scale effect) can be seen in the early part of the cloud diameter profile where the cloud shapes differ considerably.

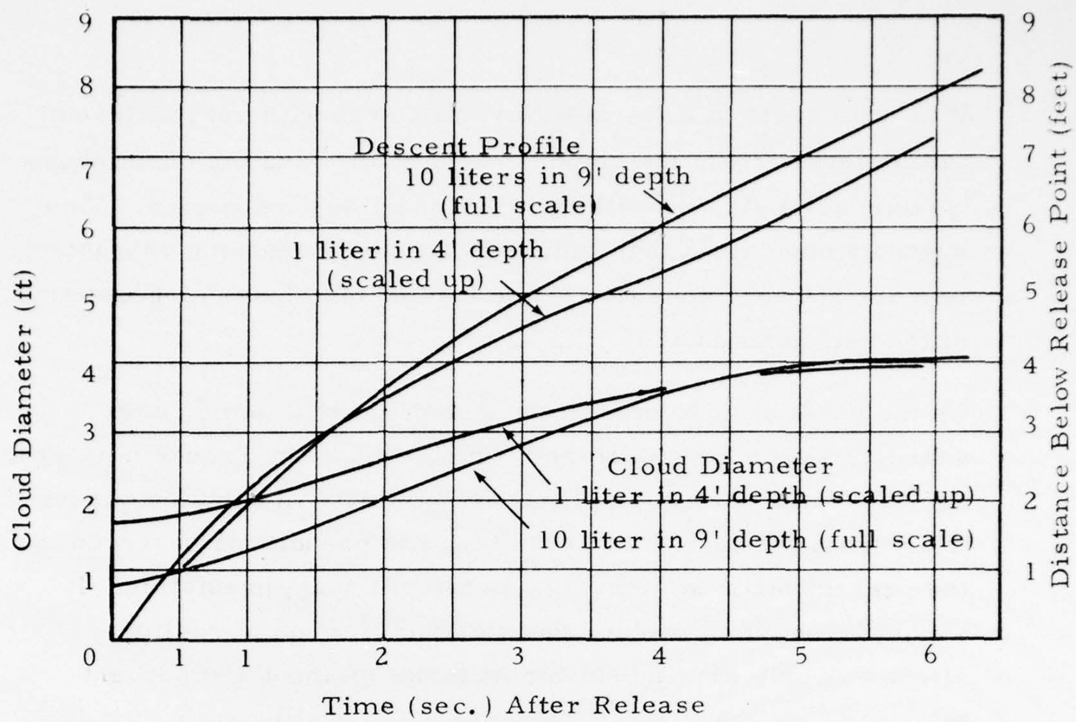



Figure 3-79. Verification of Scaling Effects - Clay

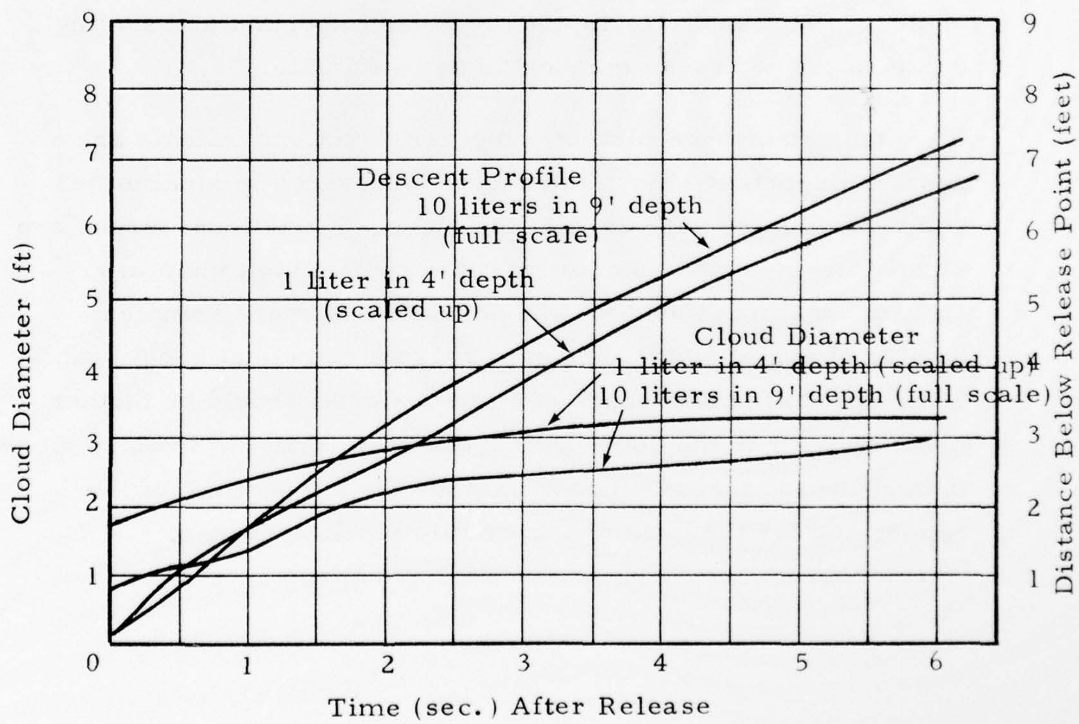



Figure 3-80. Verification of Scaling Effects - Silt

It is of interest to note, however, that even with very different initial geometries, the cloud descent profile and the cloud shape become quite similar within a very short descent period. This suggests once again that dump descent and dispersion characteristics are not very sensitive to the method of release or geometry of the releasing device.

On the basis of this comparison it appears that some dump characteristics may be scaled, using equivalent Froude numbers, and that some prediction of full scale effects in dumping can be made from the scaled test results. The phenomena observed in the present tests do form a basis for full scale prediction. It should be noted, however, that this verification of scaling effect was only carried out for material of about 190 percent moisture. At lower moisture content the scaling ratios would apply equally well since inertia effects are even more predominant when moisture content is lower and mass to area ratio even higher. Frictional effects are then less important and scaling on the basis of Froude number alone is suitable.

For higher moisture content, however, frictional effects are of greater importance since the negative buoyancy is smaller in comparison to surface area of the cloud. If frictional forces are significant in comparison to inertia forces at high moisture content, scaling effects will be considerably more complex. Scaling of 400 to 500 PCM dumps may be somewhat questionable. Scale effects for this range of dump material should be further investigated. It should be noted, however, that PCM values higher than 200 most probably occur only for the top portion of the hopper and for the case of a hydraulic pipeline dredge.

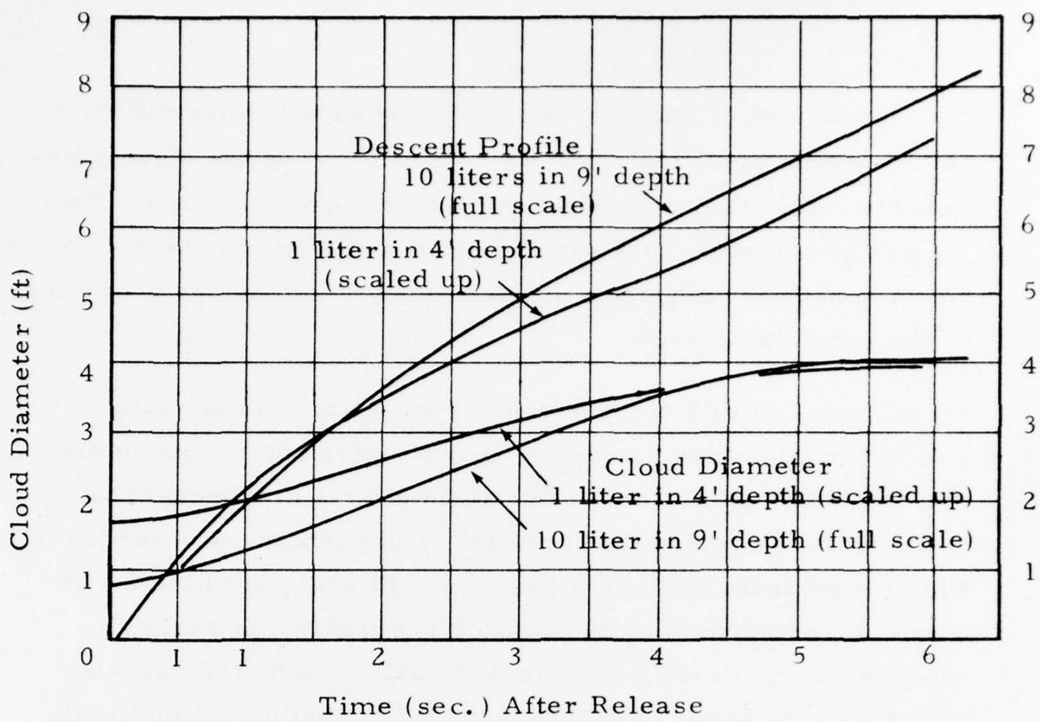



Figure 3-79. Verification of Scaling Effects - Clay

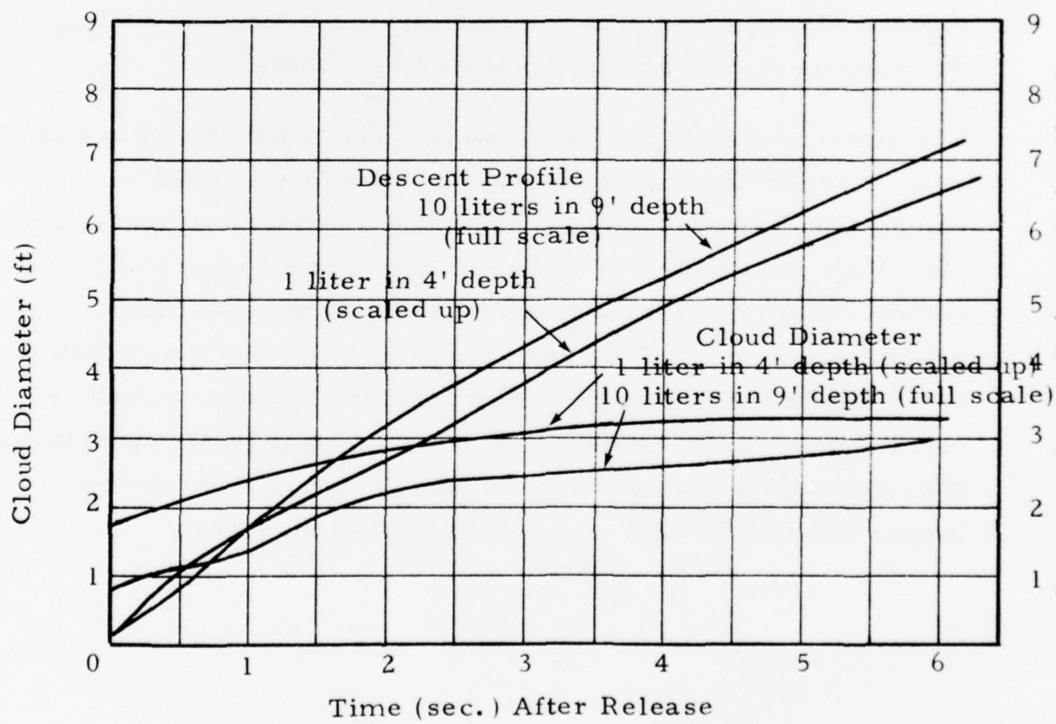



Figure 3-80. Verification of Scaling Effects - Silt

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Effects of Moisture Content

One of the major variables that affects the disposal phase is the moisture content of the material dumped. In order to evaluate the effects of this variable the following lab simulation tests were compared:

For clay material

| | | |
|---|---|---|
| Run 20 | - | 95 PCM in salt water from hopper |
| 3 | - | 121 PCM in fresh water from hopper |
| 1 | - | 131 PCM in fresh water from bag* |
| 2 | - | 135 PCM in fresh water from barge |
| 4 | - | 183 PCM in fresh water from hopper |
| 10 | - | 190 PCM in fresh water from hopper |
| 15 | - | 191 PCM in fresh water from barge |
| 22 | - | 422 PCM in salt water from hopper |
| 5 | - | 535 PCM in fresh water from hopper |

For silt materials

| | | |
|---|---|---|
| Run 11 | - | 67 PCM in fresh water from hopper |
| 6 | - | 100 PCM in fresh water from hopper |
| 19 | - | 120 PCM in salt water from hopper |
| 9 | - | 190 PCM in fresh water from hopper |
| 16 | - | 191 PCM in fresh water from barge |
| 13 | - | 403 PCM in fresh water from hopper |
| 21 | - | 410 PCM in salt water from hopper |

All of the above releases were made in a four foot water depth with 10 liters of material. Since it was found that the method of material release (barge, hopper, etc.) and the dump environment (salt or fresh water) had little effect on the dumping characteristics, it was decided to combine runs in these categories in order to obtain larger samples for recognizing trends associated with moisture content. Water depth, size of dump and type of material

* The first test run utilized a plastic bag as the dump container

(clay or silt), however, do have a significant effect on the dump and therefore these factors were held fixed for each of the above series of runs. This gave a range of 95 PCM to 535 PCM clay and 67 PCM to 410 PCM silt for the investigation.

Figures 3-81 and 3-82 show the observed effects due to changes in PCM. To characterize the descent phase the average descent velocity and the entrainment coefficient were used. These values have been plotted against moisture content for both clay and silt material in Figures 3-83 and 3-84. For both materials there is a "solid" and "liquid" range. In the "solid" range the material behaves more like a solid block with low entrainment and high descent velocity while in the "liquid" range it behaves more like a fluid cloud with higher entrainment and a relatively low descent rate. Vorticity was more likely to be generated in the descending cloud when it was in the "liquid" range. The solid range appears to extend up to about 100 PCM for both clay and silt, while the liquid range is above about 150 PCM for clay and above about 200 PCM for silt. In between these bounds is a transition range where descent velocities probably decrease and entrainment coefficients increase. It should be noted that within the liquid range for clay, the entrainment coefficient appears to increase gradually with PCM. However, for silt entrainment is essentially constant in that range and all material within the liquid range behaves similarly in descent. Caution should be exercised in attempting to define the bounds of the solid, transition, and liquid phases from this data since there is a paucity of data in the transition phase.

The effects of moisture content on the collapse phase and the bottom flow phase are also characterized by solid and liquid ranges. The flow generated along the bottom is generally high

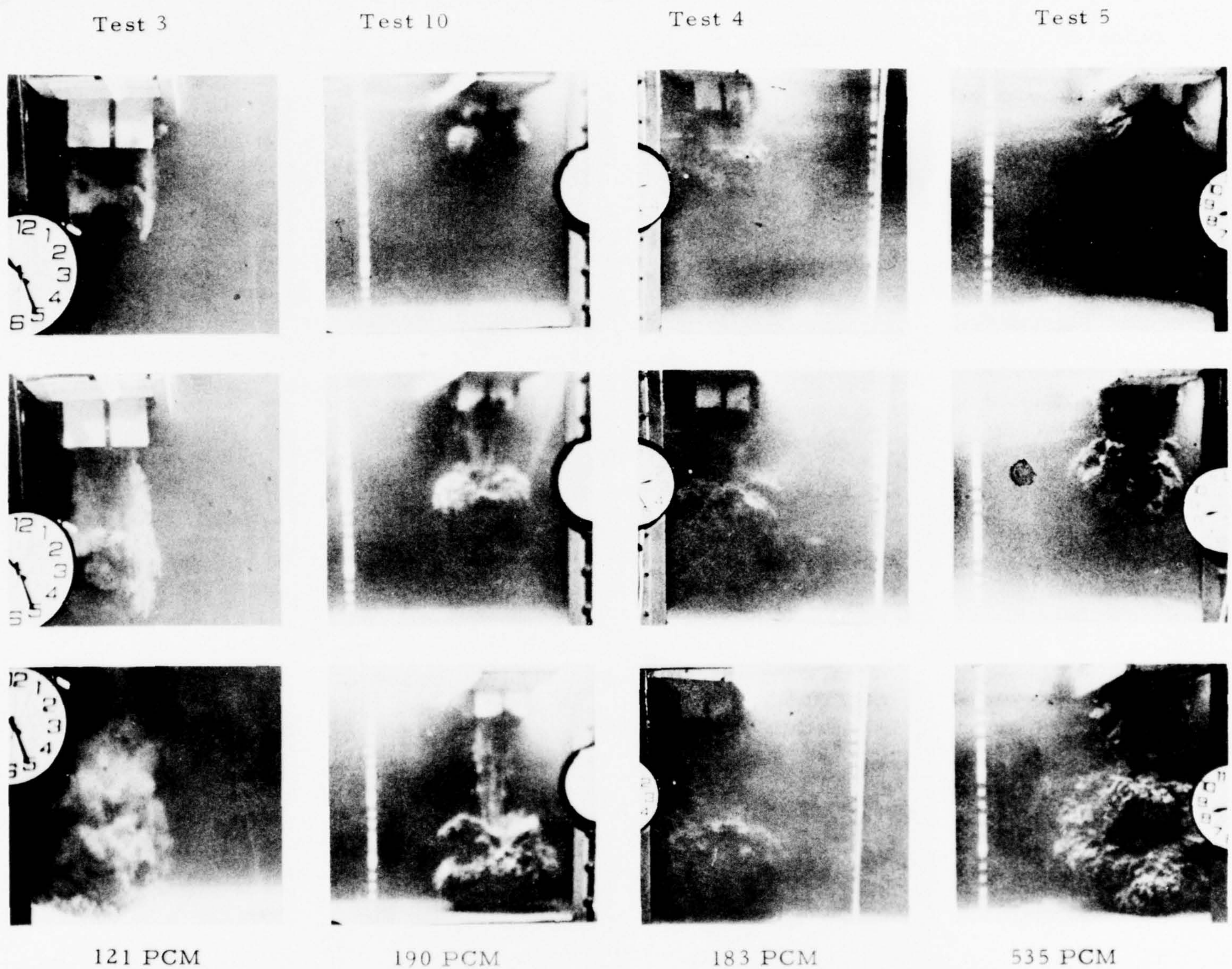
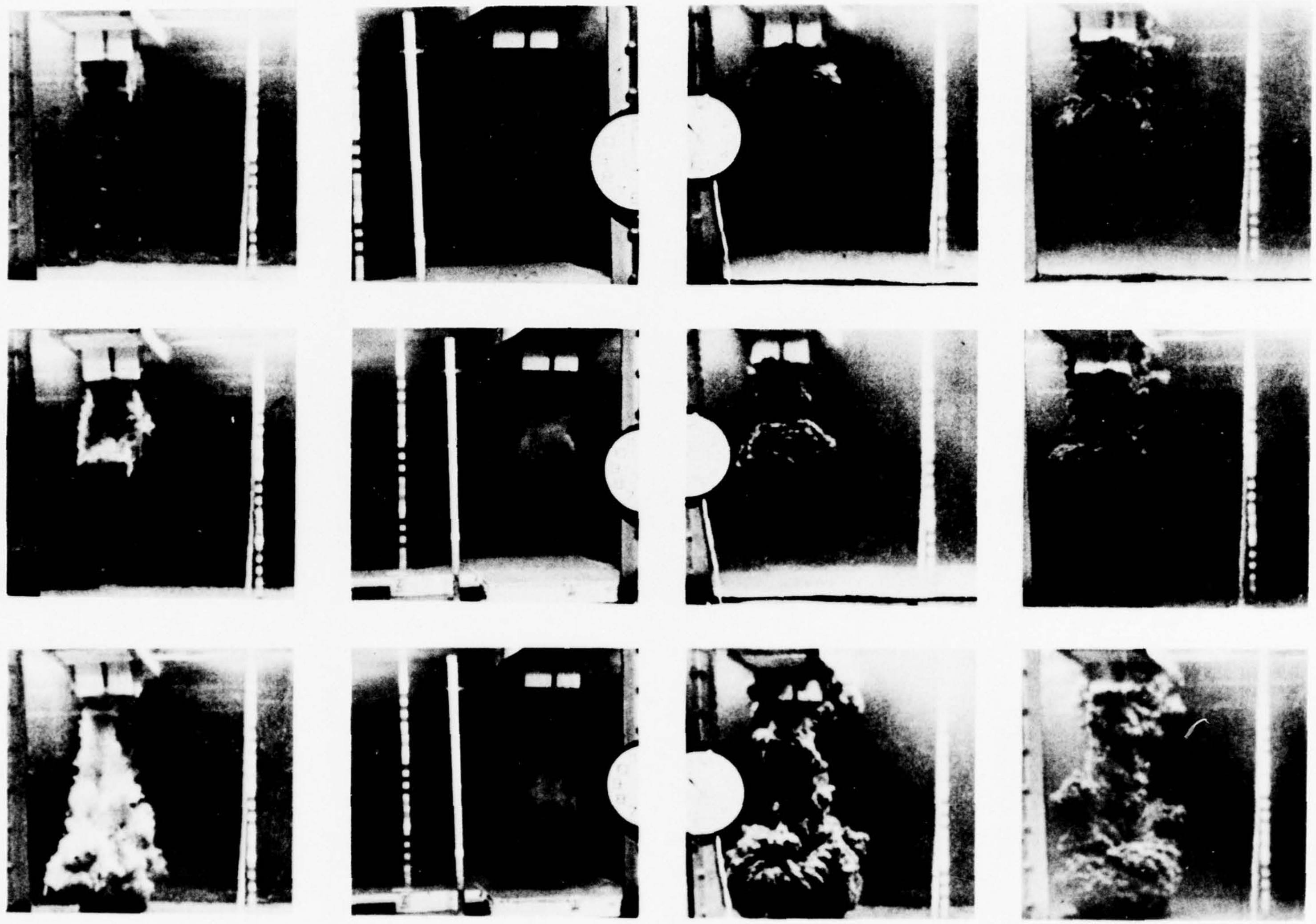



Figure 3-81. Effect of Percent Moisture on Descent Phase for Clay

Test 11 Test 6 Test 9 Test 13



67 PCM 100 PCM 190 PCM 403 PCM

Figure 3-82. Effect of Percent Moisutre on Descent Phase for Silt

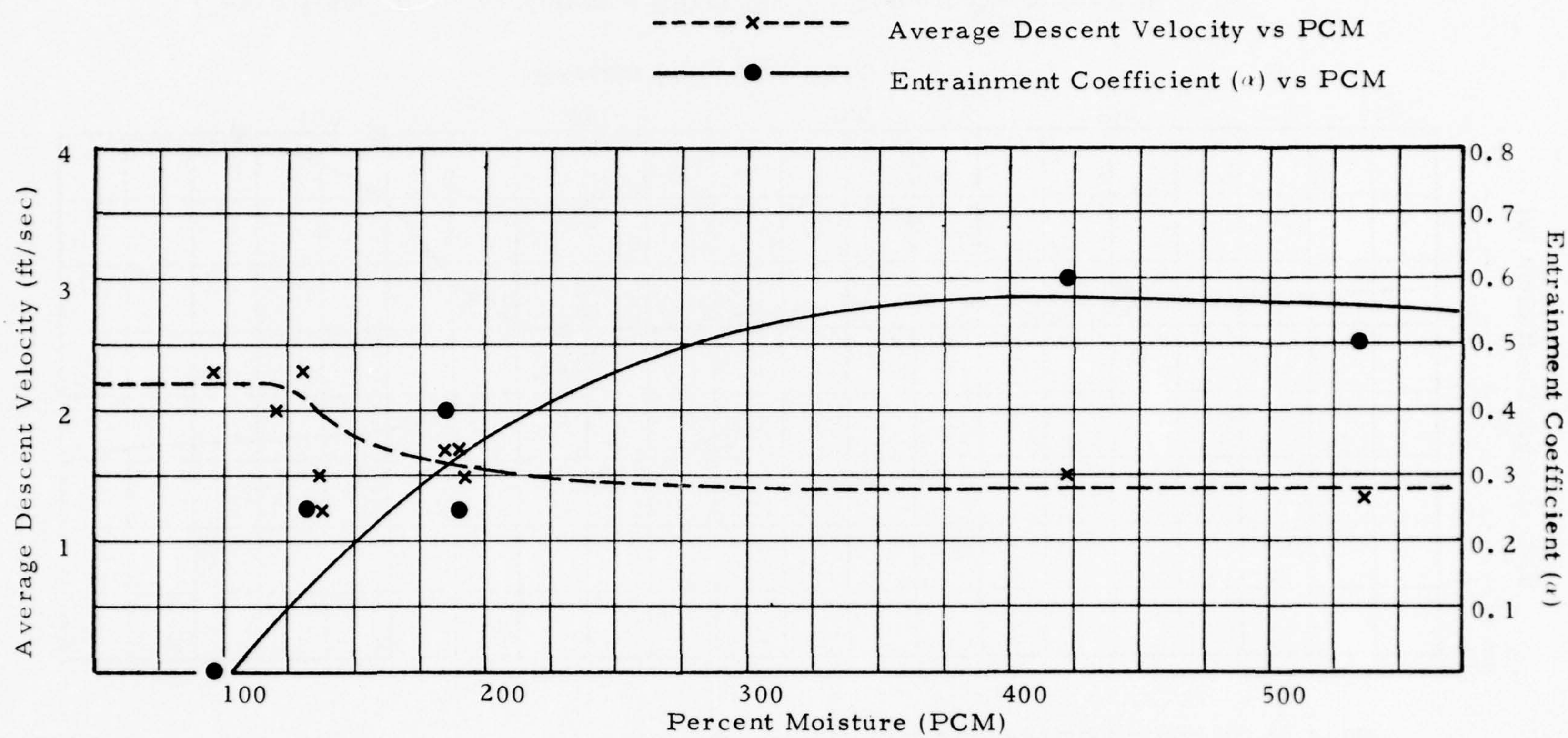



Figure 3-83. Percent Moisture Effect During Descent Phase for Clay

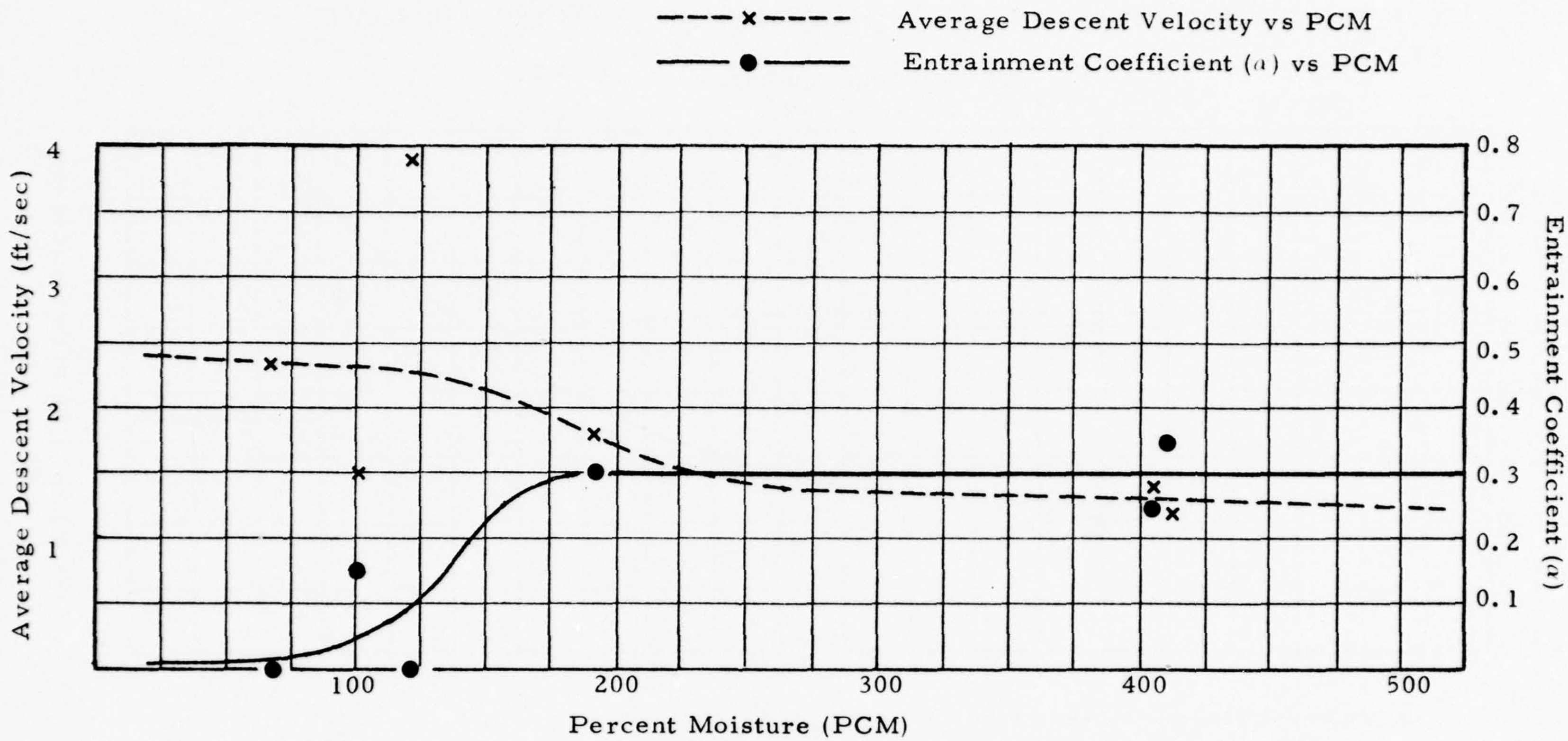



Figure 3-84. Percent Moisture Effect During Descent Phase for Silt

for dumps in the liquid range and low for those in the solid range. This is shown in Figures 3-85 and 3-86 where the average bottom flow rate in the first 6 feet from the impact point is plotted as a function of percent moisture.

During the collapse phase some of the downward momentum of the dumped material is converted to a horizontal momentum. Some measure of this effect is provided by $V_b/V_i$, the ratio of the initial bottom flow rate to the final descent velocity (just before impact). Although the measure is not ideal, since it does not take into account the fraction of material that is moved by the bottom flow, it does give an indication of the amount of impact energy that has been diverted into driving the bottom cloud. This fraction is expected to be relatively low for material in the solid range (impact energy absorbed in the bottom) and relatively high for the liquid range of dumped material (impact energy diverted to bottom flow). The ratio plotted against percent moisture, also shown in Figures 3-85 and 3-86, shows distinct solid and liquid ranges. For both the clay and silt materials the solid range appears to extend up to about 100 PCM and the liquid range is near 200 PCM. The range from 100 to 200 PCM is a transition range where the bottom flow velocity is increasing and the fraction of impact energy diverted to driving that flow is increasing. At very high moisture content (400 to 500 PCM) the bottom flow velocity appears to decrease slowly while the velocity ratio remains high. This is due to the fact that impact velocity does decrease with very high percent moisture so that the amount of momentum available on impact also decreases while the fraction diverted to bottom flow remains the same.

The influence of moisture content in the dumped material on material deposit and mound formation is also characterized by distinct solid and liquid PCM ranges. These effects are shown in

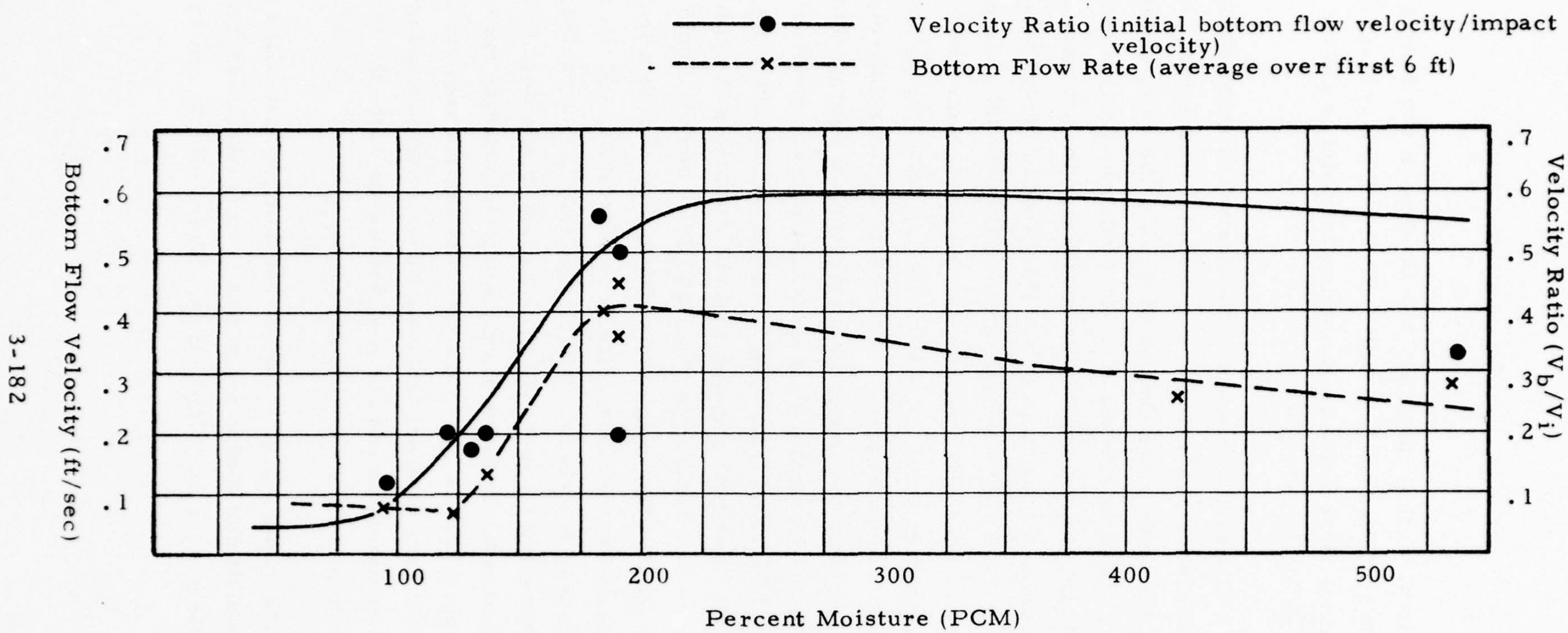



Figure 3-85. Percent Moisture Effect on Collapse and Bottom Flow for Clay

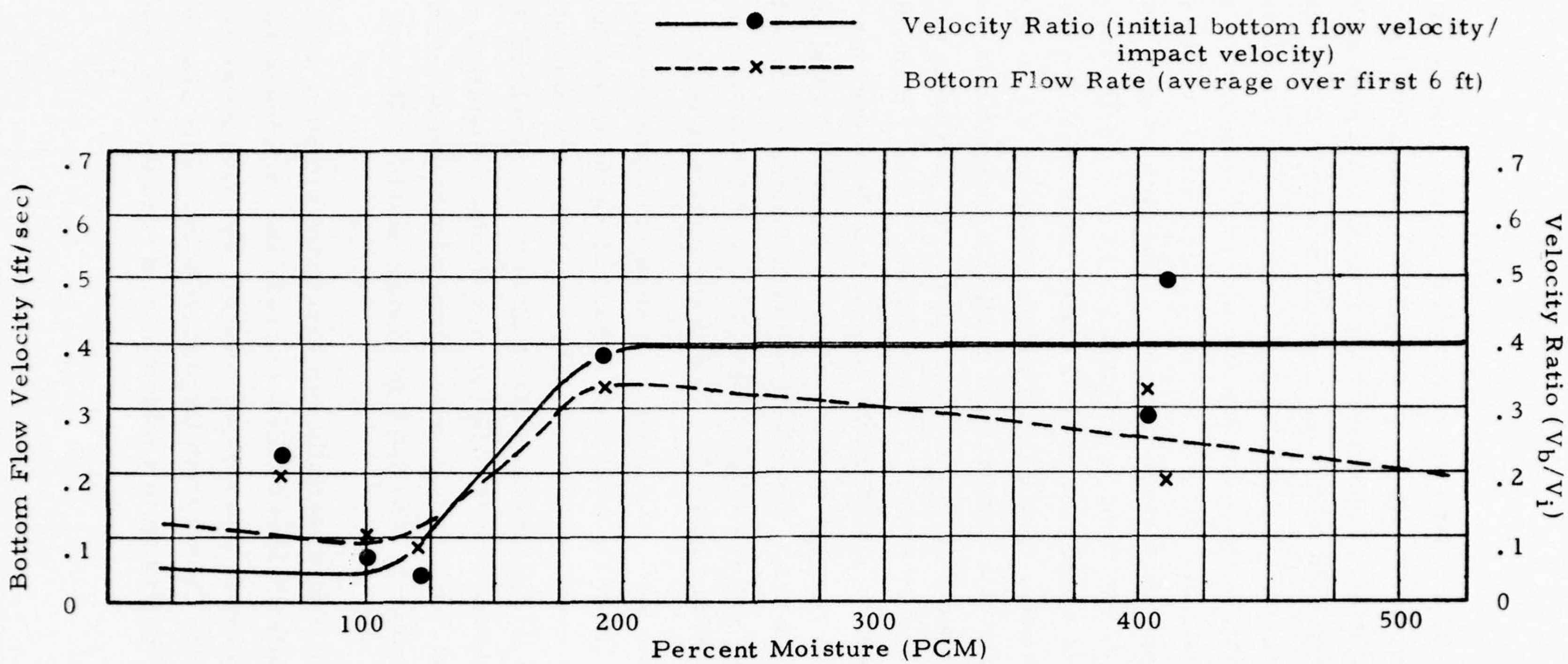



Figure 3-86. Percent Moisture Effect on Collapse and Bottom Flow for Silt

Figures 3-87 and 3-88 where mound depth and settling rate (deposition) in the vicinity of impact have been plotted against percent moisture. The mound depth is the maximum measured depth of deposited material. Deposition is an average rate of settling (mg/cm$^2$ dry weight) in the space from 2 to 6 feet from impact point. Values for deposition were determined by weighing the material which settled during each test into Petri dishes placed at specified space intervals on the tank bottom. In the solid range, extending up to about 125 PCM for both clay and silt, mounding is significant and little material is deposited away from the impact area. In the liquid range, extending beyond about 200 PCM for both materials, mounding is insignificant and greater amounts of material are apparently carried in the liquid cloud and deposited over a wider region. At very high moisture contents (400 to 500 PCM) the deposit rate decreases since nearly all material remains in suspension. Between 100 PCM and 200 PCM is a transition region where mounding becomes less predominant and a sharp increase occurs in the amount of material settling out of the cloud in the impact vicinity.

Limited data were developed in an attempt to quantify some aspects of the horizontal cloud flowing along the tank bottom. Samples for suspended solids determination were taken at one, three, and five inches above the tank bottom. Height of the advancing cloud was recorded from the motion pictures, and the level of the cloud thirty minutes after the dump was recorded. Several problems were noted in connection with collecting these data:

- To obtain spatially representative samples, 1/4" polyethylene tubing was used, and samples were gently pumped into sample bottles. The tube openings were set 3.5 feet from the impact point. This procedure required up to a minute, and there may have been some

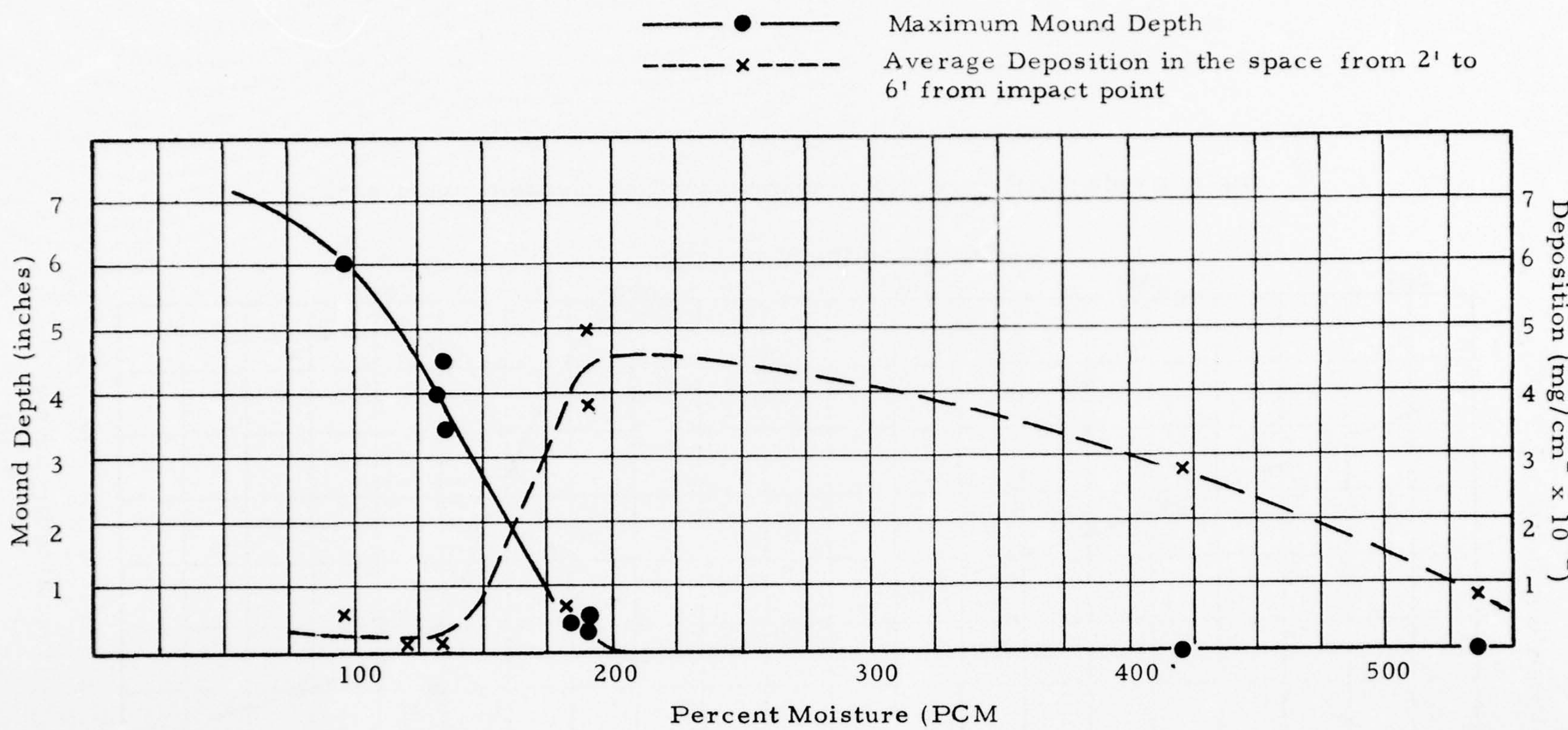



Figure 3-87. Percent Moisture Effect on Mounding and Deposit of Clay

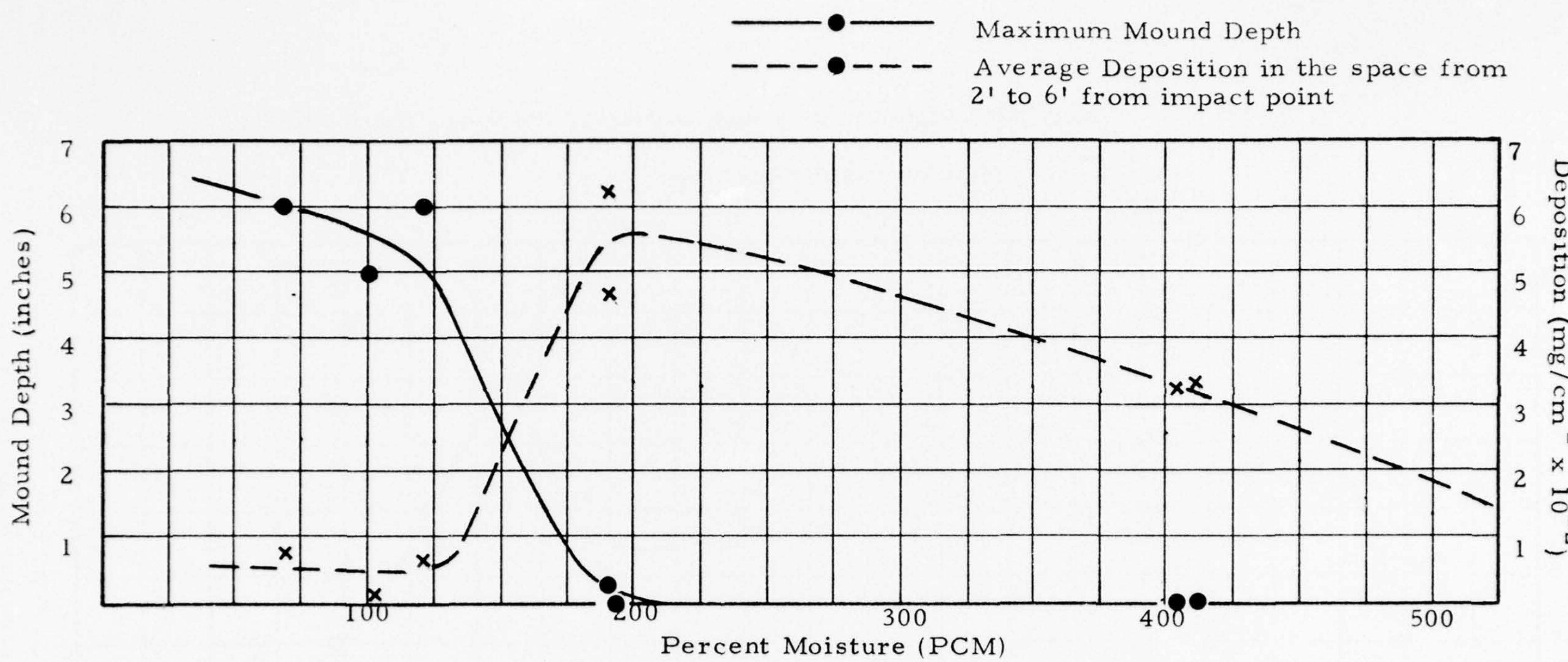



Figure 3-88. Percent Moisture Effect on Mounding and Deposit of Silt

effect from the cloud's impact and reflection from the fartank wall.

- Cloud height and ultimate cloud level are affected by the constraint of tank size.

These observations on horizontal cloud movement are tabulated in Table 3-23, and some data are plotted in Figure 3-89. The importance of moisture content is again indicated by this plot, which suggests that cloud solids may be fairly constant (insensitive to PCM of dumped material) above 200 PCM for silt in fresh water. The figure also shows the higher concentration of suspended solids near the bottom. Other very general trends can be inferred from Table 3-23. Silt appears to generate a cloud lower in suspended solids than clay, and clouds in salt water appear to have lower suspended solids than clouds in fresh water.

Two-layer dump test

As indicated earlier, a single test was run to observe the effect of percent moisture layering, or gradient, in a dump vessel. A sample was prepared using 5 liters of 100 PCM on the bottom and 5 liters of 400 PCM material on top. It was postulated that the 100 PCM material would fall quickly to the bottom and that the 400 PCM would fall more slowly. If this occurred it implies that the descent phase from a hopper dredge would consist of a series of material layers, each behaving in a manner related to the PCM and volume of the various layers. In actuality, there may not be discrete "layers" but perhaps layers with transition zones between them.

Figure 3-90 shows the two-layer test photos. In the left photo the 100 PCM block of material can be seen, with material

TABLE 3-23

Data on Cloud Moving Horizontally Across Tank Bottom

All Tests with 10 Liter Dumps in 4 Feet of Water

| Test No. | Water (Fresh/ Salt) | Release (Hopper/ Barge) | Type (Clay/ Silt) | PCM | Susp. solids at Height off bottom (mg/l) 1" | 3" | 5" | Max. Wave Height (ft.) | Final Cloud Height (in.) |
|---|---|---|---|---|---|---|---|---|---|
| 9 | F | H | S | 190 | 1504 | 447 | 452 | 1.4 | 11 |
| 10 | F | H | C | 190 | 2906 | 712 | 389 | 1.1 | 6 |
| 11 | F | H | S | 67 | 99 | 65 | 33 | 2.9 | 7 |
| 13 | F | H | S | 403 | 1778 | 1050 | 454 | 0.8 | 10 |
| 15 | F | B | C | 191 | 2220 | 1604 | 1111 | 1.9 | 9 |
| 16 | F | B | S | 191 | 1722 | 1272 | 565 | 1.0 | 8 |
| 17 | F | H | S | 187 | 122 | 23 | 13 | 1.5 | 4 |
| 19 | S | H | S | 120 | 36 | 44 | 38 | 2.1 | 4 |
| 20 | S | H | C | 95 | 31 | 53 | 37 | 0.8 | 8 |
| 21 | S | H | S | 410 | 1022 | 370 | 154 | 2.0 | 12 |
| 22 | S | H | C | 422 | 1059 | 620 | 278 | 0.8 | 14 |

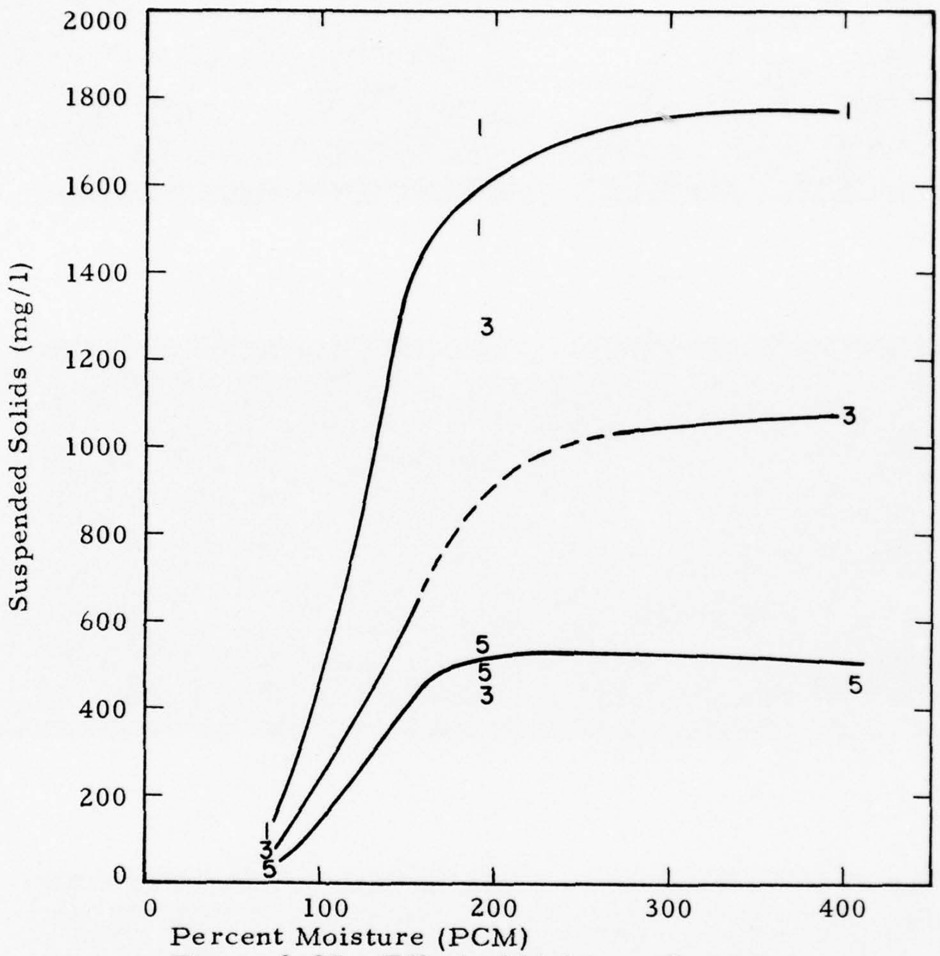



Figure 3-89. Effect of Moisture Content on Suspended Solids in Horizontal Plume. Plotted Numbers Refer to Inches Above Tank Bottom.

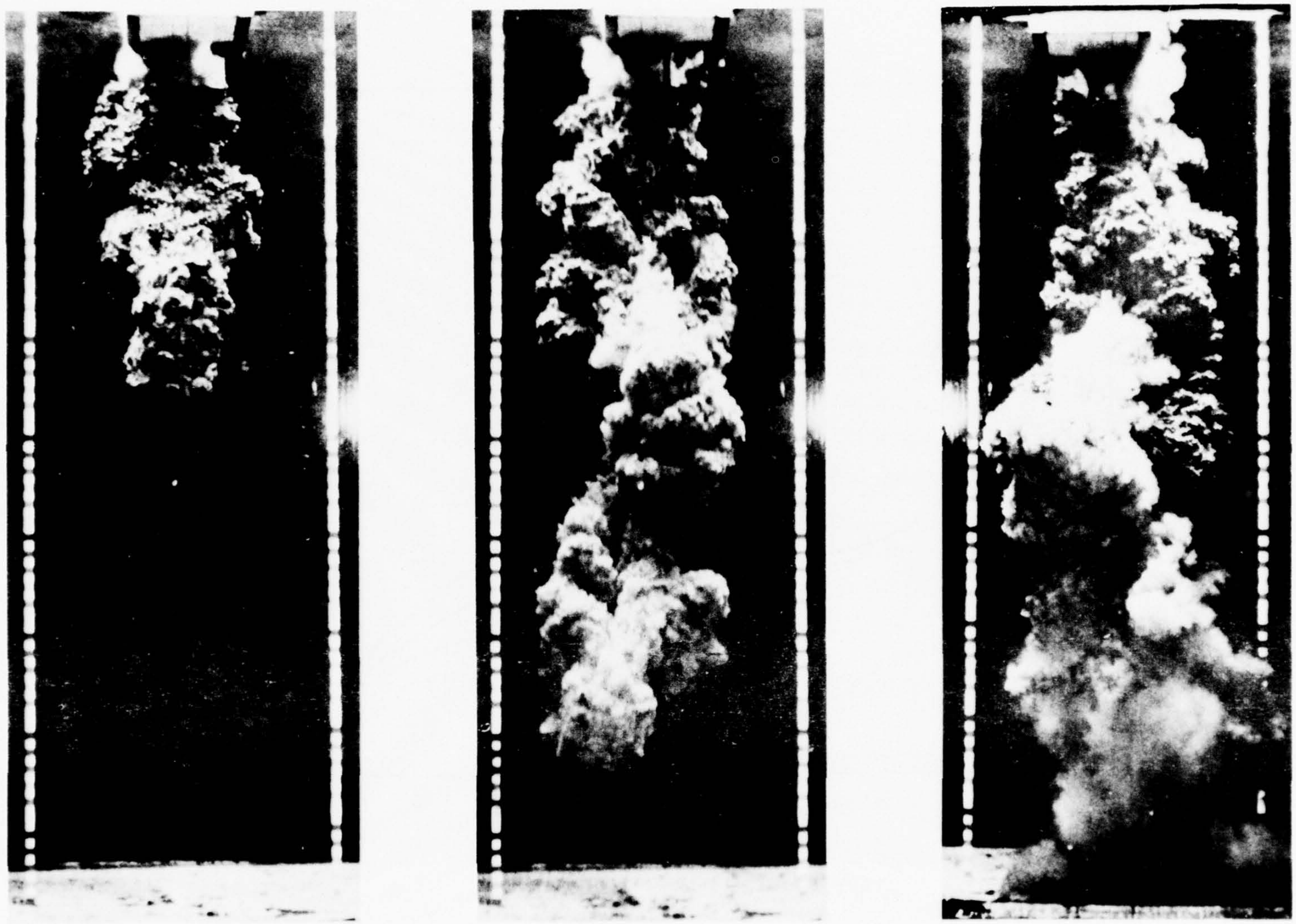


Figure 3-90. Two-Layer Drop of 100 PCM and 400 PCM Material (Test 26)

streaming off of it, into its wake. Just leaving the hopper is a dark cloud that signifies the exit of the 400 PCM material. In the center photo the 100 PCM block, still intact is about to impact on the bottom. Note that the distance between the leading edge of the 100 PCM material and the dark cloud of 400 PCM material has increased considerably.

In the photo on the right, the 100 PCM material has impacted and generated an impact cloud similar to these seen on other drops of material with a low percent moisture. The dark cloud is seen to be descending at a much lower velocity.

Examination of the movies of this dump, as compared to movies of 100 PCM and 400 PCM dumps, indicate that the gross characteristics are the same, especially for the 100 PCM fraction, but that the flow behind the 100 PCM material does affect the behavior of the 400 PCM material following it. Thus, there is an interaction that must be considered, when predicting the behavior of a stratified dump, and simple superposition may not be adequate. However, the entrainment coefficient for the 400 pcm layer is about the same as encountered in other high-percent moisture tests.

(3) Interpretation of Observed Effects in Large Tank Tests

On the basis of the observations described in the preceding section, two important hypotheses are suggested. The first is that some discharge and dumping characteristics can be scaled and predictions to full scale made on the basis of equivalent Froude number. The second hypothesis is that, for the materials studied, clay and silt, dumps can be classified on the basis of percent moisture into "solid" or "liquid" PCM ranges and that, within each classification, the dumping characteristics are relatively independent of the material's percent moisture, Between the "solid" and "liquid" ranges there is a transition range where these characteristics are highly variable and are dependent on percent

moisture. The size of this transition range may be a function of the depth, size of the dump, and the type of material.

Validity of Froude Number Scaling

When a hydrodynamic phenomenon is driven principally by inertia effects and friction effects are negligible in comparison to the inertia effects, that phenomenon may be modeled or scaled on the basis of Froude number. That is, when both the model and the full scale have the same Froude number, they will behave similarly and model characteristics may be scaled to full size using scaling relationships defined by the Froude number.

In the case of ocean dumping it appears likely that inertia effects predominate when the dump behaves as a cloud rather than as individual particles. Thus, during the descent phase and perhaps much of the bottom flow phase, scaled tests may be used to predict full scale behavior. When diffusion predominates, or settling takes place, friction effects and individual particle behavior is also important and then unique scaling relationships cannot be determined. Once in the diffusion phase where individual particle behavior is the predominant effect, scaling is no longer appropriate except on the basis of individual particle dimensions.

The hypothesis that inertial effects predominate in the descent phase and that modeling based on equivalent Froude numbers is therefore appropriate, has been supported by comparisons between geometrically similar dumps of differing depth and size. These comparisons indicated that in the "solid" PCM range and at least in the lower part of the "liquid" PCM range, small scale tests may be extrapolated to full scale on the basis of Froude number. To do this, the small scale behavior may be observed

in a tank where water depth and hopper dimensions are reduced in a suitable geometric scaling factor ( $\ell$ ). The volume of dumped material is then reduced by the cube of that scaling factor ($\ell^3$). The observed time or velocity of descent and bottom flow may then be expanded back to full size by the square root of the scaling factor $\sqrt[2]{\ell}$ ; cloud or mound dimensions by the factor ( $\ell$ ) and cloud or mound volumes by the factor cubed ( $\ell^3$).

Extrapolation to Full Scale

The contract statement of work requires that "Qualitative predictive conditions shall be extrapolated to actual transport modes and distances and disposal site conditions."

In this section the scaled study results will be extrapolated to typical full scale conditions utilizing three predictive approaches. These are:

- the Krishnappan mathematical model
- the Koh Chang mathematical model
- Froude number scaling of actual test dumps

The Koh Chang model is based primarily on theoretical considerations while the Krishnappan model is based largely on empirically derived relationships.

As a result of the present study, a third method of predicting dump behavior has been suggested. This is the expansion of small scale test data to full scale. This method has the advantage of not requiring precise quantitative definition of all significant properties of the dredged material. For example, although cohesiveness of dredged material may have a significant influence on the dump characteristics, it is presently not possible to account for this in the analytical models except as it can be defined in terms of the resulting particle sizes. However, from the scaled tests of the present study, the importance of material

cohesion was readily apparent.

In the following comparisons between analytical and small scale test methods of prediction, it will be shown that analytical methods predict considerably slower descent velocities than those predicted by the expansion of small scale data. In the case of the Krishnappan model it will also be shown that model predictions, applied at the reduced scale test dimensions, did not correspond to the observed descent characteristics. The differences appear to be due to the influence of material cohesion on descent characteristics.

Krishnappan Model

In the development of his model, B. B. Krishnappan has taken a dimensional approach to formulate the basic relationships in the descent phase of dumping. He has then used laboratory simulation dumps to empirically determine the required coefficients for growth and descent velocity.

His basic relationships are the following:

$$w = \frac{\beta\left[\left(\frac{\rho_s - \rho}{\rho}\right) g\, V\right]^{1/2}}{z + Ro/\alpha} = \text{cloud descent velocity} \qquad (3\text{-}1)$$

and

$$R = Ro + \alpha z = \text{cloud radius}$$

where $\rho_s$ is density of the dumped material

$\rho$ is density of the ambient water

$g$ is acceleration of gravity

V is volume of dumped material

z is depth at which velocity is measured

Ro is initial radius of dumped volume

$\beta$ and $\alpha$ are empirically derived coefficients that are dependent on the value:

$$\frac{(\rho_s - \rho)\, g\rho\, D^3}{\mu^2}$$

where D is the particle size of the material dumped

$\mu$ is the viscosity of the ambient water.

These relationships are the core of the Krishnappan model. In addition, the model provides for a settling phase where material settles at individual particle fall velocities but, for the purpose of comparing simulation test results with Krishnappan predictions, for the materials under study, this settling phase can be ignored since it can be shown that the cloud will impact the bottom before it slows sufficiently for individual particle settling to occur.

The Krishnappan model was applied to a number of the simulation test conditions, using as input, a specific gravity of 3.0 for the solid material and using an initial radius (Ro) based on a spherical initial volume. In these comparisons, even the largest particle sizes in the clay and silt material (about .04 mm diameter) were too small to settle as individual particles within the depth rnage under consideration. Thus, only the entrainment phase of the model was applied.

The results of this comparison are shown in Table 3-24. Cloud radius is quite well predicted for 400 and 200 PCM moisture content, however, at 100 PCM the model considerably over-estimates the radius. Impact velocity is considerably under-

TABLE 3-24 - Comparison of Scaled Test Tank and Krishnappan Model Predictions

| Dump Conditions | | | Cloud Radius on Impact | | | Impact Velocity | | |
|---|---|---|---|---|---|---|---|---|
| Moisture Content (PCM) | Dump Volume (liters) | Water Depth (ft) | Krishnappan Model $r_m$(ft) | Test Tank $r_t$(ft) | Ratio $r_t/r_m$ | Krishnappan Model $V_m$ (ft/sec) | Test Tank $V_t$ (ft/sec) | Ratio $V_t/V_m$ |
| 400 | 10 | 4 | 1.4 | 1.0 | .71 | .32 | .8 | 2.5 |
| 200 | 1 | 4 | 1.2 | .8 | .67 | .15 | .5 | 3.3 |
| 200 | 10 | 4 | 1.4 | 1.2 | .86 | .44 | 1.6 | 3.6 |
| 200 | 10 | 9 | 3.0 | 1.9 | .63 | .20 | .7 | 3.5 |
| 100 | 10 | 4 | 1.4 | .5 | .36 | .58 | 2.5 | 4.3 |
| 100 | 10 | 9 | 3.0 | .7 | .23 | .27 | 3.0 | 11.1 |

estimated at all levels of PCM although the prediction appears to improve with increasing moisture content. Mounding predictions are poor as all PCM levels were not tabulated.

Probably the major factor accounting for these differences between model predictions and actual test results is the material cohesiveness. The Krishnappan model was developed for a non-cohesive material (sand) and the materials used in the simulation tests (clay and silt) were highly cohesive. We did, however, use the Krishnappan empirically derived constants for the particle size of interest in these tests. In general, cohesive material has been demonstrated to fall more rapidly and show less cloud growth than non-cohesive material. In the low moisture content tests (100 PCM), for example, the material actually fell as a solid block with nearly no cloud growth. Its drag and entrainment were low and hence its impact velocity was high. However, the Krishnappan model predictions, based on non-cohesive material (but with the correct particle size) overestimated cloud radius by a factor of 2 and underestimated impact velocity by a factor of more than 6.

With higher moisture content materials, cohesiveness is less important and thus model prediction and test results appear to converge.

In considering the prediction of full scale effects by means of the Krishnappan model, it should be noted that, under model predictions for the descent phase, scaling is dependent only on the Froude number. This can be shown as follows:

From equation 3-1

$$w = \frac{\beta\left[\left(\frac{\rho_s - \rho}{\rho}\right) g V\right]^{1/2}}{z + Ro/\alpha}$$

Taking $\ell$ as a characteristic length where:

$$\ell = z + Ro/\alpha$$

then, for equivalent geometry, volume is proportional to length cubed:

$$V = \gamma \ell^3$$

so that:

$$w = \frac{\beta \left[ \left( \frac{\rho_s - \rho}{\rho} \right) g \gamma \ell^3 \right]^{1/2}}{\ell}$$

squaring both sides and transposing:

$$\frac{w^2}{gl} = \beta \left( \frac{\rho_s - \rho}{\rho} \right) \gamma$$

and $\frac{w^2}{gl}$ is the Froude number.

This indicates that, under the model assumptions, Froude number is dependent only on dump characteristics(( $\rho_s$, $\rho$, $\beta$ ) and the dump geometry ( $\gamma$ ). Thus, the Krishnappan model predictions for a reduced scale dump (for the entrainment phase) are expanded to full scale model predictions on the basis of Froude number. This is the same basis that we used in the expansion of the scaled test data to full scale. Therefore, a comparison between full scale predictions with the Krishnappan model and full scale predictions based on expansion of the test tank data will be equivalent to the comparison of reduced scale effects and the same conclusions will apply.

## Koh Chang Model

The Koh Chang model is a computerized prediction method that has been developed from extensive theoretical considerations, as described previously. For the purpose of a comparison with scaled test tank results, however, only the convective descent phase will be considered. This phase takes account of cloud drag, negative buoyancy, friction forces and entrainment in calculating the growth and trajectory of the descending cloud.

In order to compare the Koh Chang predictions with simulation test results, the Koh Chang model has been run for various full scale conditions of dumping, and appropriate test tank results have been expanded to equivalent scale. The expansion of the test tank data is on the basis of Froude number as described previously. The results of these comparisons are shown in Table 3-25.

As in the previous comparison the mathematical model appears to overestimate cloud growth and underestimate the descent velocity, although the discrepancy is not as large as in the comparison with the Krishnappan model. Again, the most probable explanation lies in the cohesiveness of the dredged material. The materials used in the scaled tests were highly cohesive and, therefore, tended to have little entrainment and a very rapid descent velocity while the entrainment coefficient suggested in the Koh Chang model is probably based on a less cohesive material. However, it appears that the entrainment coefficient suggested for use in the Koh Chang model is better suited to the types of material under study than that provided by the Krishnappan model and thus, the model provides a somewhat better fit to the test data. The discrepancy between the scaled test data and the Koh Chang prediction appears to increase with decreasing moisture content. This is expected since cohesiveness

TABLE 3-25 - Comparison of Koh Chang Predictions With Test Tank Data

Scaled to Full Scale Using the Froude Number

| Dump Conditions | | | Cloud Radius on Impact | | | Impact Velocity | | |
|---|---|---|---|---|---|---|---|---|
| Moisture Content (PCM) | Dump Volume (liters) | Water Depth (ft) | Koh Chang Model $r_m$ (ft) | Scaled Test Data $r_t$ (ft) | Ratio $r_t/r_m$ | Koh Chang Model $V_m$ (ft/sec ) | Scaled Test Data $V_t$ (ft/sec) | Ratio $V_t/V_m$ |
| 200 | 310 | 50 | 29.8 | 15 | .50 | 9.9 | 11 | 1.1 |
| 100 | 310 | 100 | 41 | 15 | .36 | 9.8 | 14 | 1.4 |
| 100 | 40 | 50 | 20.6 | 8 | .39 | 6.9 | 10 | 1.5 |

is of greater importance with lower PCM material.

The fact that the Koh Chang predictions are closer to the scaled test predictions than those made using the Krishnappan model is not necessarily a consequence of the greater theoretical complexity of the Koh Chang model. The improved fit appears to reflect a choice of entrainment coefficient in the Koh Chang model that is better suited to the materials used in the scaled tests.

Expansion of Test Tank Data to Full Scale

The major weakness in both of the analytical models discussed above is in the difficulty of adequately representing characteristics of the dump material. The effects of cohesion, for example, cannot be modelled without first empirically defining characteristics, such as the entrainment coefficient and the liquid limit, for the specific material of interest.

The prediction of dump characteristics by means of scaled tests avoids this difficulty since the material under study is actually used in the scaled tests. The expansion of test tank data to full scale, however, is a problem that requires further consideration. On the basis of the present study, it appears that much of the test tank data can be expanded on the basis of Froude number. However, certain aspects of mud flow and diffusion may not be equivalently scaled. Scaling relationships for all dump phases should be further investigated and verified by laboratory and field tests.

Another variable which is difficult to simulate in dumping tests is specific equipment configuration. Within normal time and budget constraints, perspective would be lost by simulating the many vessels which might be used for bottom dumping in the Bay. Hopper dredges, barges, and scows all have dumping mechanisms

and shapes unique to each vessel. The data resulting from these dumping tests are strictly applicable only where the shape of the pocket opening of the full-scale vessel approximates the shape of the opening in the test equipment. Application to dumping of multiple pockets, or to equipment with unusual configurations, should be done with caution and with an awareness of these potential problems.

For the present study, test tank data for the descent phase has been expanded on the basis of Froude number to provide a set of curves relating impact radius and velocity to dump size and water depth. These relationships appear in Figures 3-91 to 3-98.

In preparing these curves the tests in the 9 foot tank were used. These tests were with clay and silt, with each at about 100 and 200 PCM moisture content. The actual cloud descent profile was used to derive descent velocities and cloud radii at various depths and these were expanded to various full scale dump volumes and water depths on the basis of equivalent Froude numbers. The radii and velocities so determined may be interpreted as the impact velocities and radii for the equivalent water depth even though the measurements may not correspond to the actual impact point in the scaled test. The justification for this is in the observation made previously in this chapter that descent profile is not dependent on depth of water. That is, for example, with equivalent dump conditions the descent profile over the first 4 feet of a 9 foot drop is equal to that of an entire 4 foot drop. Since there is no apparent effect due to bottom proximity, the characteristics of the dump cloud at some intermediate depth in the drop, may be considered equivalent to the impact characteristics for a drop in that limited water depth.

Some anomalous results were noted for the 50-foot depth calculations. These anomalies were caused by the fact that scaling

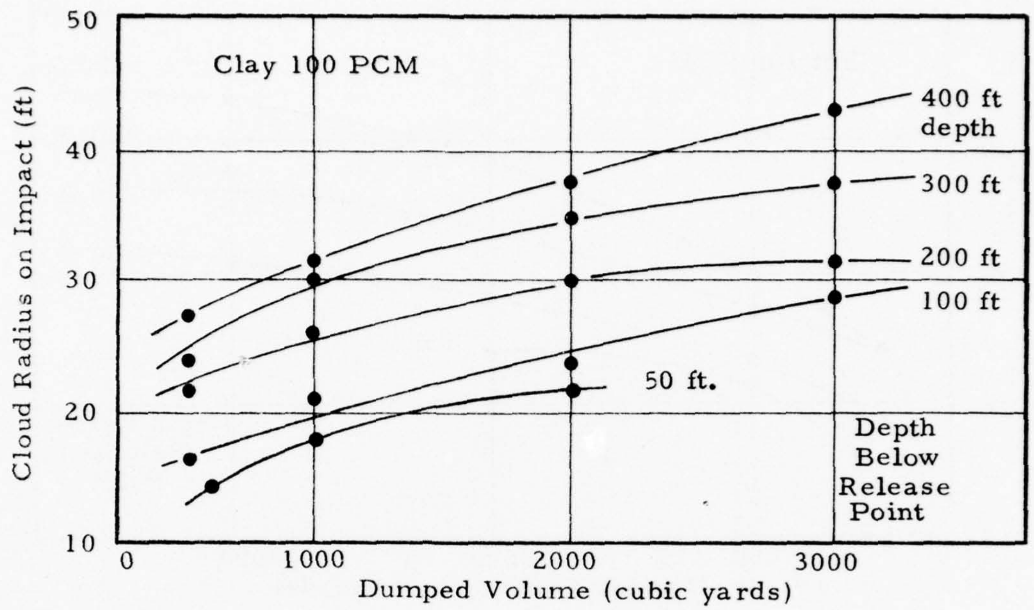



Figure 3-91. Full Scale Cloud Radius on Impact

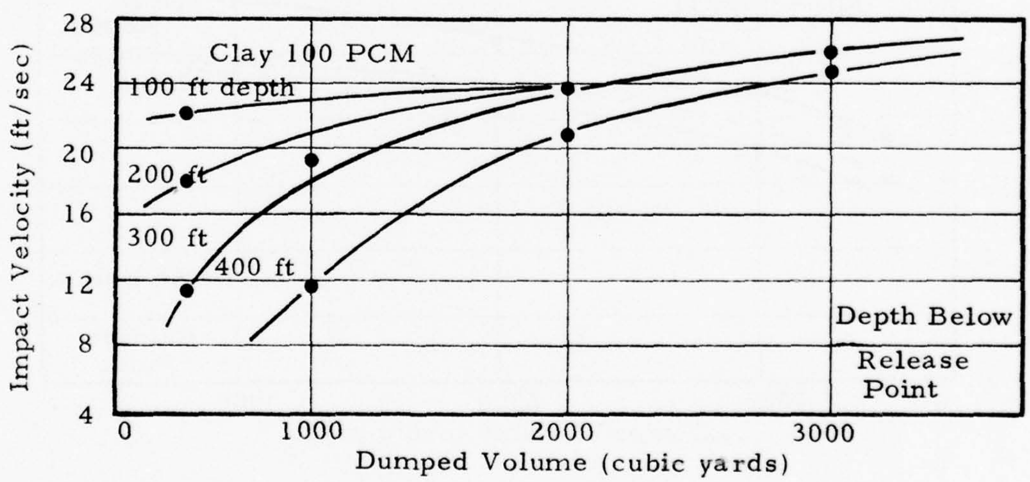



Figure 3-92. Full Scale Impact Velocity

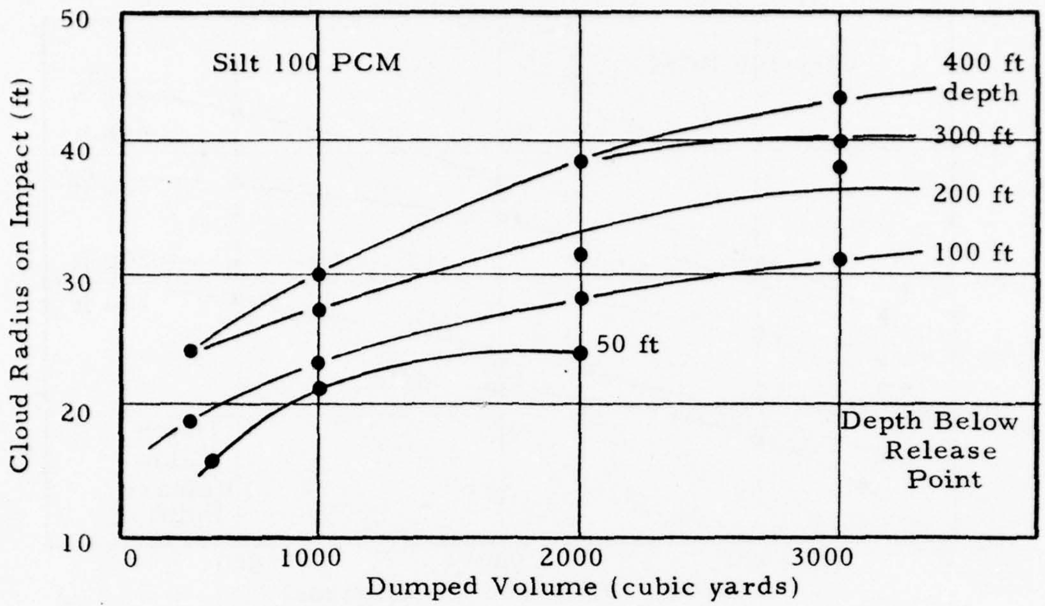



Figure 3-93. Full Scale Cloud Radius on Impact

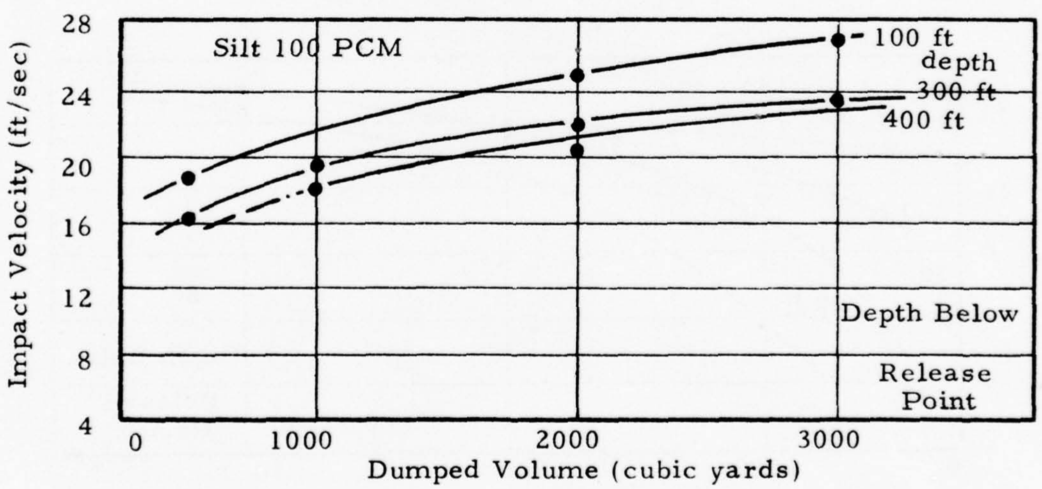



Figure 3-94. Full Scale Impact Velocity

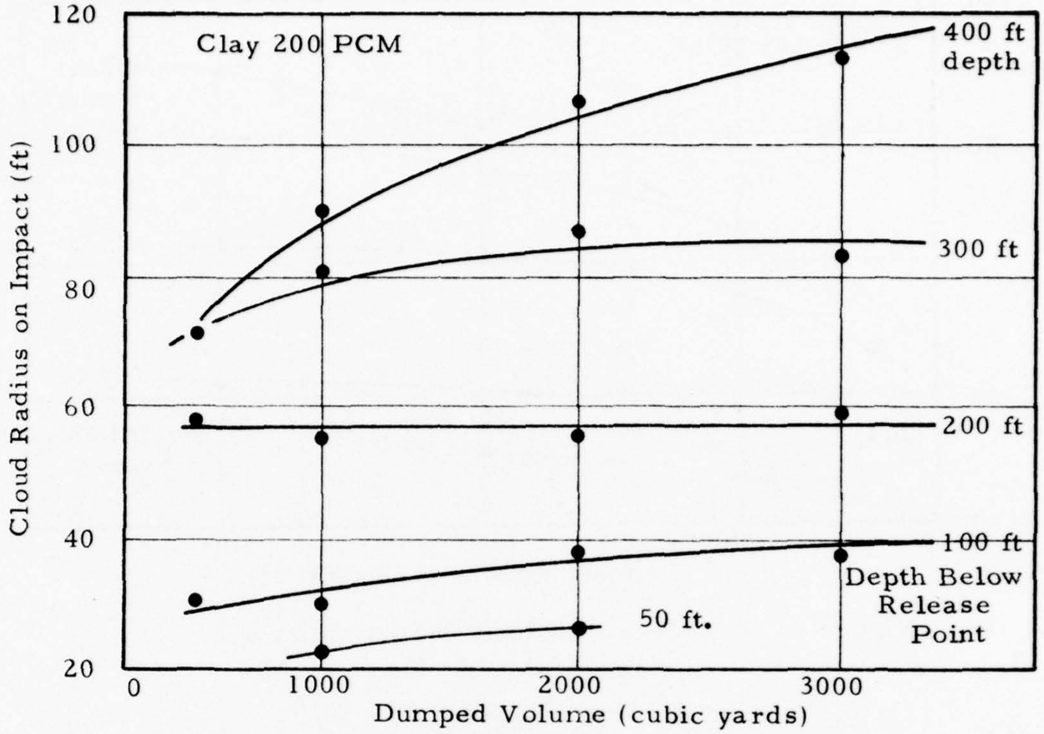



Figure 3-95. Full Scale Cloud Radius on Impact

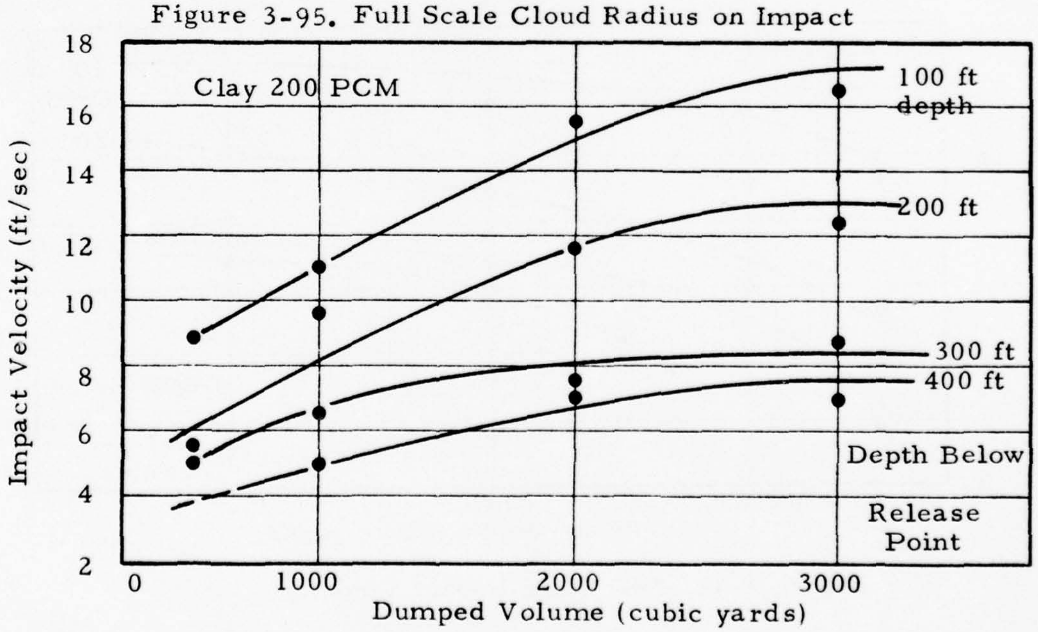



Figure 3-96. Full Scale Impact Velocity

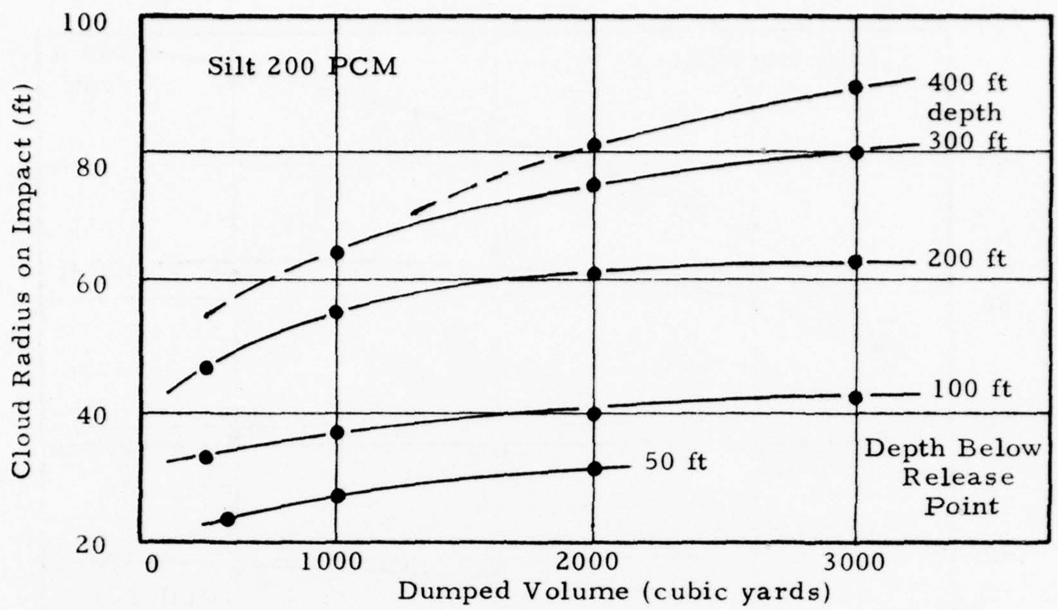



Figure 3-97. Full Scale Cloud Radius on Impact

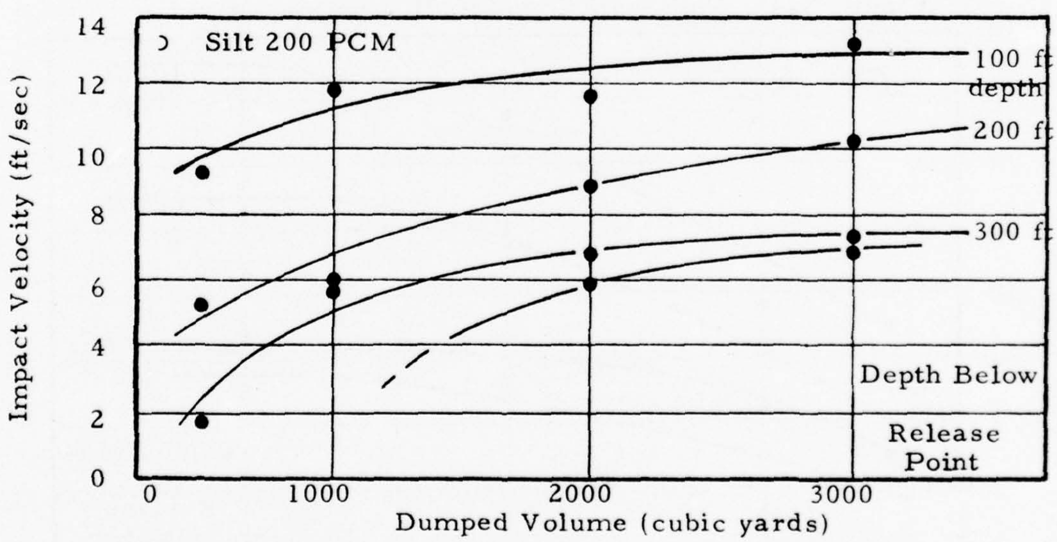



Figure 3-98. Full Scale Impact Velocity

to the 50-foot depth relied on data from the first two feet below the release point. This is a region where transient effects, unique to the specific release configuration, predominate. Therefore, the values for 50-foot depth on Figures 3-91 to 3-98 are incomplete.

In using these curves it should be kept in mind that they are tentative estimates at this time since only limited verification of the scaling relationships has been carried out. However, since the Krishnappan and Koh Chang models both scaled using the Froude number, it would appear that the best prediction method for full scale San Francisco Bay open water disposal would be to use these curves that were generated from actual Bay sediments in scaled tests and extrapolated using Froude numbers.

Using the full scale curves in Figures 3-91 to 3-98, and the measured and inferred behavior of the cloud, as described in previous sections, it is possible to make quantitative predictions of the cloud radius and impact velocity for 100 PCM and 200 PCM Bay sediments such as found in Pinole Shoals and Mare Island Straits. It is also possible to then make qualitative predictions of the bottom flow, mounding and cloud settling characteristics for these materials based upon the data presented in this chapter. Finally, the behavior for material dumped from a bottom dumping barge, or a hopper dredge, can be inferred from the field measurements and laboratory tests presented in this chapter. The full scale curves for 100 PCM materials represent material in a bottom dumping barge, or the high density, low moisture content of a hopper dredge. The 200 PCM curves represent the behavior of material in the low density, high moisture content of a hopper dredge.

## Percent Moisture as a Controlling Parameter

The test results described in this section suggest that, for the materials under study (clay and silt), dumping characteristics fall into two distinct modes. The "solid" mode is characteristic of materials with low percent moisture such as seen in the barge filled with a clamshell in Alameda and in the bottom half of the hopper dredge in Mare Island. In this mode the dumped volume falls as a solid block or blocks and does not spread much on the bottom. In the "liquid" mode, characteristic of materials with a high moisture content such as the top few feet in the hopper dredge, or in a pipeline dredge, the dumped material falls as a liquid cloud and spreads like a fluid on the bottom. These modes can be identified by relatively well defined PCM ranges although the cut-off PCM values for each are dependent upon the size and depth of the dump. The "solid" range includes all PCM values below a certain transition point (upper bound of solid mode), and the liquid range includes all PCM values above a somewhat higher transition point (lower bound of liquid range). The PCM range between these two points is a transition range where dumping characteristics vary rapidly with change in PCM and dumps show characteristics of both modes. Although there is some variation in dumping characteristics by PCM <u>within</u> each mode, the differences between modes are much greater and the transition between these modes takes place over a relatively narrow transition range of PCM.

The "solid" dump mode is characterized by a very rapid descent phase, little cloud growth and little spread of the material on the bottom after impact. During descent the material falls like a heavy, dense block trailing a turbidity plume behind it. It rapidly reaches its equilibrium velocity and does not decelerate

before impact. Entrainment is negligible in the solid mode so that the main cloud has little shape change during descent as illustrated in Figure 3-99. Most of the descent energy is absorbed on impact and does not contribute to spreading material along the bottom. Thus, the bottom flow is low in suspended solids and is slow moving. Most of the dump material is deposited in a mound at the impact point. There is a distinct possibility that the behavior of the material just after impact, for the "solid" range, is related to the cohesiveness of the material.

The "liquid" dump mode also shown in Figure 3-99 is characterized by a slower descent phase with the cloud expanding due to entrainment, and by a rapid flow of material along the bottom after impact. During descent entrainment is significant and the cloud grows rapidly, decelerating while it does so. Impact velocity may not be as high as for equivalent "solid" dumps. Most of the impact energy is redirected to a horizontal momentum that drives the cloud rapidly across the bottom. There is little or no mounding of dumped material at the impact point and most deposited material is carried in a rapidly moving bottom flow.

On the basis of a limited amount of test data, comparing dumps from 1/10 to 10 litres in size and from 16 inches to 106 inches of water depth, it appears that the PCM bounds for "solid" and "liquid" dump modes may be dependent on the dump size and the water depth. In general, it appears that with increasing depth or decreasing dump size the lower bound of the liquid range is decreasing. That is to say, for a percent moisture such that the material behaves, as if it were in the transition range for some size and depth of dump, it may be within the liquid range when dropped in deeper water or when dropped at smaller volumes, due to entrainment causing the PCM of the cloud to increase.

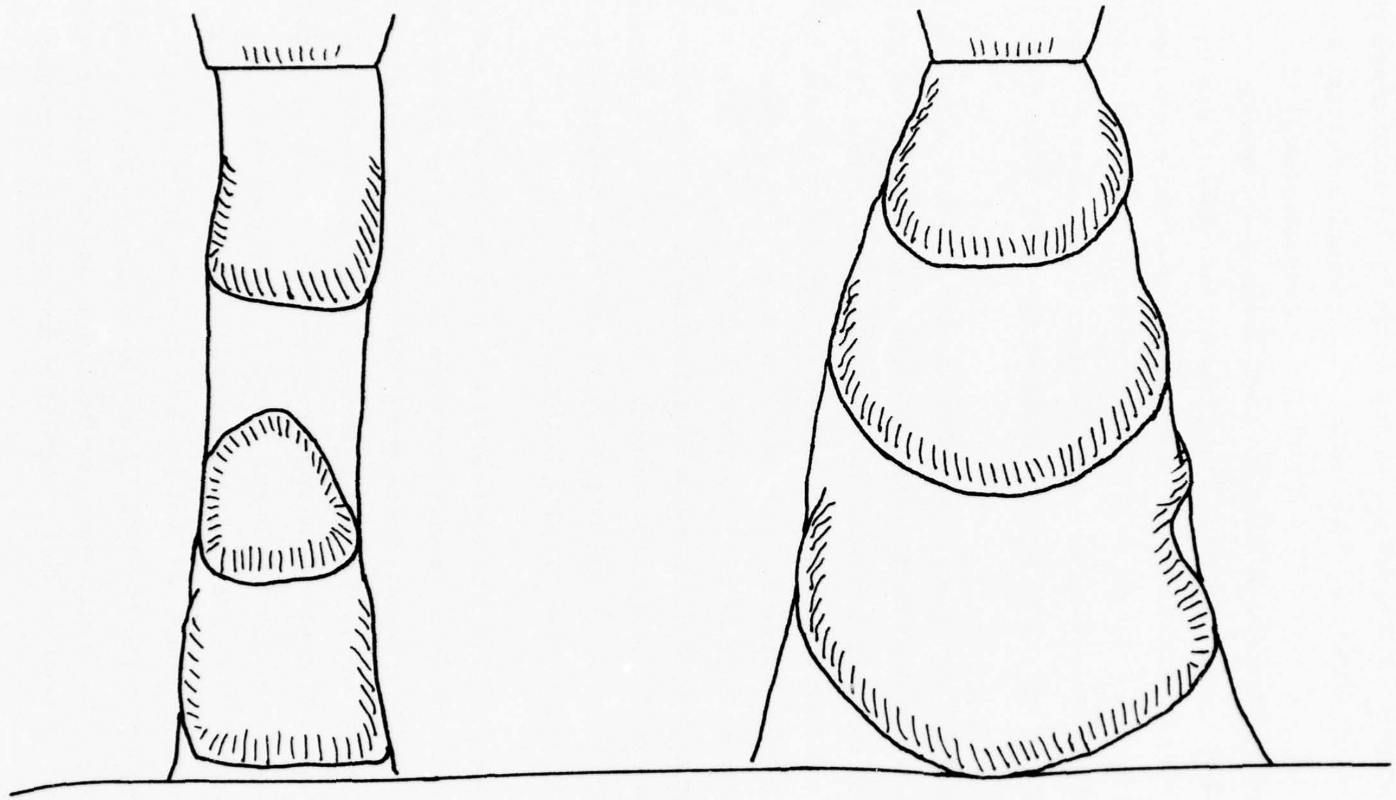



Figure 3-99. Schematic of Descent Phase for Low PCM and High PCM Materials

The upper bound of the solid range, on the other hand, appears less variable with dump size and depth.

One clue to a possible interpretation of these observations is given by the observed relationship between entrainment coefficient and percent moisture. In general, it appeared from tests that a coefficient of zero (no entrainment) would hold for percent moisture up to about 100 PCM for both silt and clay. Beyond that PCM level, the entrainment coefficient increased with percent moisture to reach a level of about 0.25 or 0.30 for silt of PCM about 200 or greater, and a level of about 0.5 for clay of PCM greater than about 300.

Given such a relationship between percent moisture and entrainment, it appears that for any dumps of low moisture content (under 100 PCM) entrainment will be negligible throughout descent and moisture content of the impacting cloud will be essentially the same as that at release. For high percent moisture drops, on the other hand, (over about 200 for silt, over about 300 for clay) entrainment will be significant but at a constant rate throughout descent so that, on impact, the cloud will have reached a significantly greater moisture content than it had at release. For an intermediate moisture content, however, entrainment becomes a dynamic phenomenon. In the course of descent, entrainment causes dilution of the descending cloud to a greater PCM. The greater PCM, in turn, allows a higher entrainment coefficient and thus a faster entrainment and dilution of the cloud. This process will result in an accelerating rate of entrainment <u>as long as the cloud PCM remains in the transition range.</u> Provided that water depth is great enough or cloud size small enough this accelerating entrainment will eventually increase the cloud's moisture content to the point that it enters the "liquid" range

and, by the time of bottom impact, its behavior may have become characteristic of the "liquid" dump mode.

This description of entrainment and descent behavior is tentative since it is based on limited data on PCM effect as a function of cloud size and water depth. However, if verified it does provide a physical basis for the observed "solid," "liquid" and "transition" dump modes. For dumps with moisture content below the critical PCM value the entrainment coefficient will be zero and entrainment will not occur so that, regardless of depth or size of dump, descent will be in the "solid" mode. Above that value, however, the descent mode will be dependent on dump size and water depth. For high ratios of depth to size, materials released at initial PCM in the transition range may have sufficient descent time to reach the liquid PCM range (by entrainment) before impact, while for low depth to size ratio, only material released at relatively high initial PCM will reach the liquid range before impact. Thus, a small depth to size ratio would be associated with a long transition range and a relatively high bound to the "liquid" range. At large depth to size ratios a shorter transition range and a lower bound to the "liquid" range would be expected.

Again it must be emphasized that the hypotheses presented here must be considered as tentative physical explanations of the observed phenomena. In support of these hypotheses, however, are, first, the observed relationship between entrainment coefficient and moisture content and second, the observation that, for dumps with initial PCM in the transition range, entrainment coefficient does indeed tend to increase during descent.

For the materials under study (clay and silt) estimates of the PCM ranges for "solid," "liquid" and "transition" dump modes have been made. These are based on limited data from the

large tank tests and these were not designed to examine a wide range of moisture contents in combination with variations of dump depth and size.

d. Tests in Small Tanks

While conducting the experiments in the large tanks it was observed that mounding of solids on the tank floor was closely related to the percent moisture in the dredged material. Significant differences in behavior were noted with changes in percent moisture for both silt and clay. A series of tests were developed to investigate the relationship between percent moisture and the extent and nature of mounding for both of the material types previously used in the large tank experiments.

Tests were performed in an aquarium with surface dimensions of 30 inches by 12 inches by 18 inches deep with a capacity of about 28 gallons. During the tests the water depth was maintained at 16 inches. Dredged material dumps of 100 ml occured slightly below the surface by manually sliding a sheet of plastic away from the face of a 2-inch diameter cylinder. This method of release was found to be simple and effective, although movies revealed that the sliding action did slightly affect the initial behavior of the material in some cases. As the dredged material settled, motion pictures, color slides, and black and white photographs were taken to document the descent through the water and degree of mounding on the bottom. To assist in interpreting the pictures aluminum meter sticks were positioned on both sides of the dump point along the centerline of the tank.

The materials dumped were termed silt and clay and had the following characteristics:

| | Silt | Clay |
|---|---|---|
| percent - sand | 13 | 2 |
| silt | 57 | 53 |
| clay | 30 | 45 |
| Original (unmixed) PCM | 96 | 169 |
| Volatile Solids, percent | 5.5 | 7.4 |
| Liquid Limit(PCM) | 47 | 78 |

The two materials are seen to be fairly similar with the clay having particles of somewhat smaller size, more organic matter, and a considerably higher original water content. Figure 3-100 and 3-101 show dispersed and non-grain size analysis for the 2 materials.

A few minutes after each dump the water level in the aquarium was lowered with a pump to approximately one inch so that overhead photographs and measurements of the mounds could be made. Description of the mounds included both lateral extent and depth of deposit where appropriate. The percent moisture of the dredged material was varied by adding appropriate amounts of tap water and mixing with a laboratory stirrer. A portion of each sample was later tested to determine its percent moisture (PCM). A description of each test dump follows.

Silt - 96 PCM

This drop was at the original sample water content of 96 PCM. The dredged material did not break up during descent, but moved toward the front of the tank during descent as shown in Figure 3-102. No dispersion resulted and the mound had a height of 3.9 cm.

Silt - 108 PCM

The dredged material broke up somewhat during descent and it appeared that if the depth were greater, the solids would have

| SAMPLE NO. | SYMBOL | DEPTH | LL | PI | UNIFIED CLASSIFICATION |
|---|---|---|---|---|---|
| JBF Aquarium Sample #562 "Clay" Dispersed | ——————— | | | | |
| JBF Aquarium Sample #562 "Clay" Non-Dispersed | — — — — | | | | |

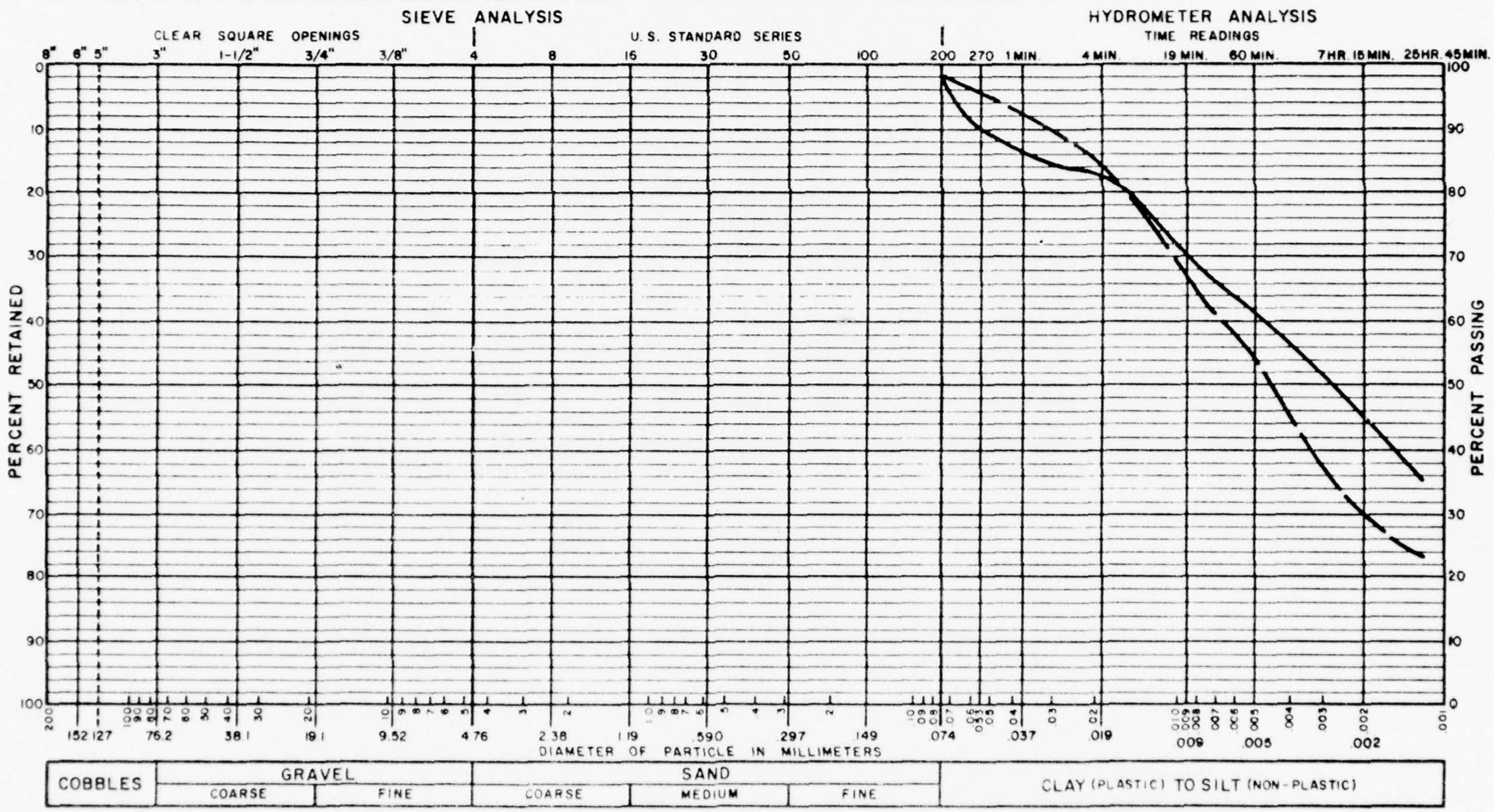



Figure 3-100. DISPERSED & NON-DISPERSED GRAIN SIZE ANALYSES - JBF AQUARIUM CLAY

| SAMPLE NO. | SYMBOL | DEPTH | LL | PI | UNIFIED CLASSIFICATION |
|---|---|---|---|---|---|
| JBF Aquarium Sample #561 "Silt" Dispersed | | | | | |
| JBF Aquarium Sample #561 "Silt" Non-Dispersed | | | | | |

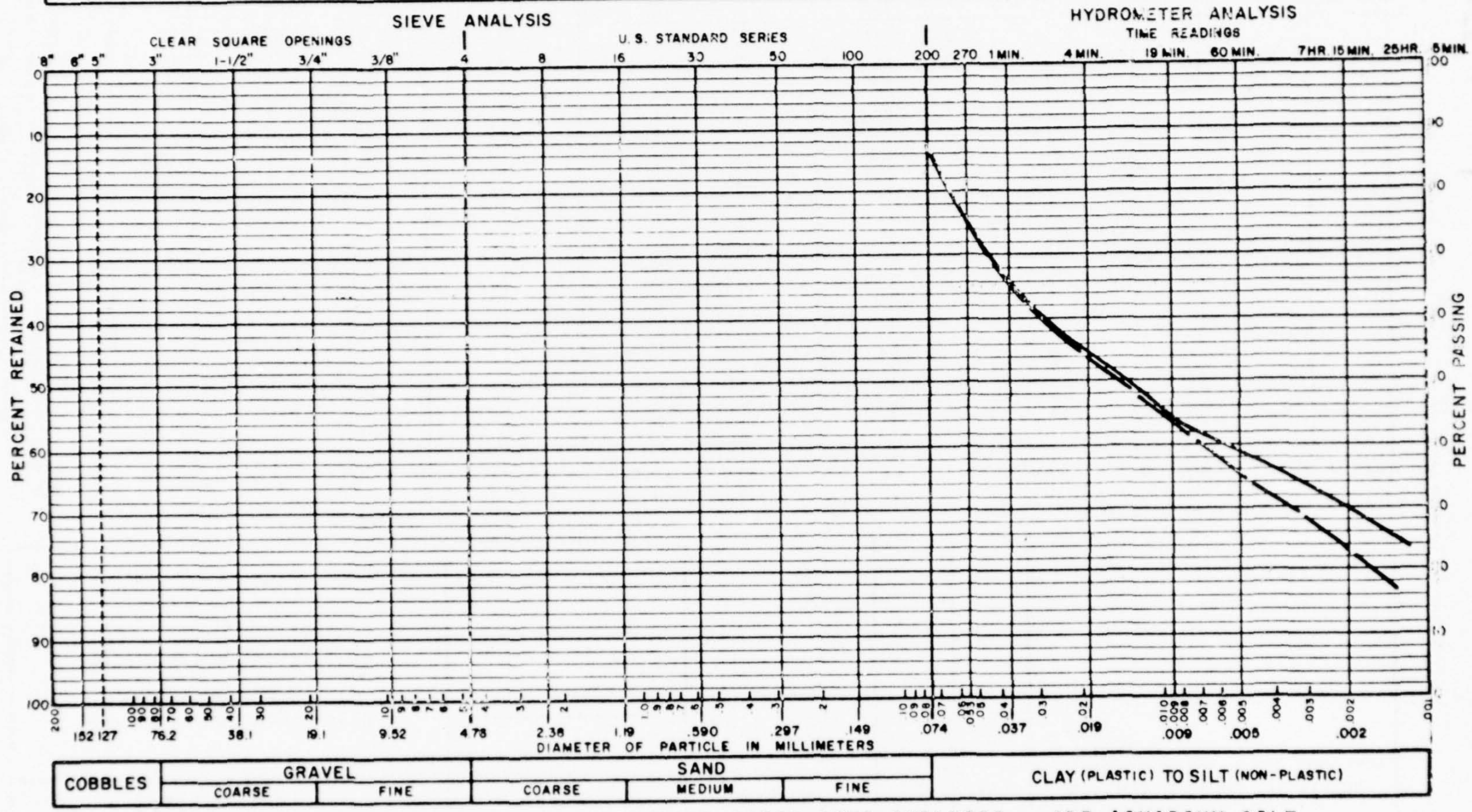



Figure 3-101. DISPERSED & NON-DISPERSED GRAIN SIZE ANALYSES - JBF AQUARIUM SILT

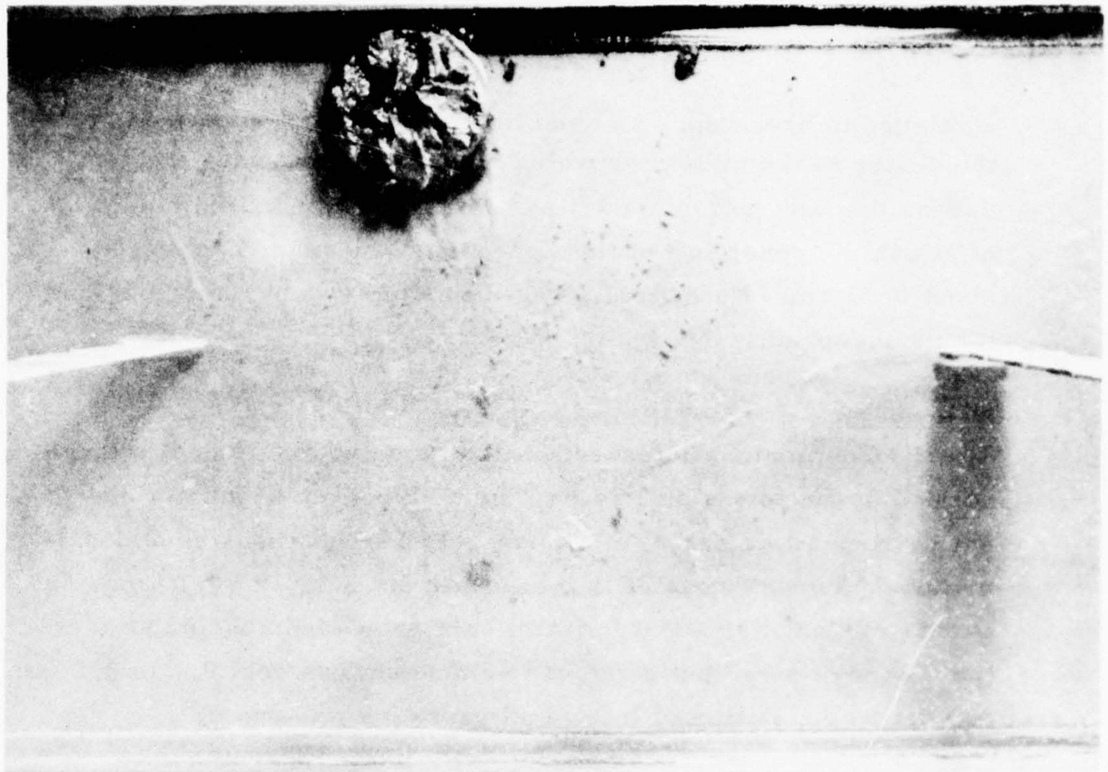


Figure 3-102 Silt - 96 PCM

Figure 3-103 Silt 108 PCM

continued to break up. As seen in Figure 3-103, impact was at the center of the tank after which most of the material slid across the tank bottom as a single mass. The maximum depth of silt in the upper left of the figure was 1.1 cm and averaged about 0.75 cm. No material was deposited except where obvious lumps occurred.

Silt - 114 PCM

The dredged material descended in a cloud and on impact a large portion of the solids slid across the tank bottom to form a ring shaped deposit as shown in Figure 3-104. The ring was approximately 30 cm in diameter with a maximum height of 0.7 cm. The average height was about 0.5 cm. Material also accumulated at the center of ring, but depth of this deposit was only 0.1 to 0.2 cm. Almost no depositing of individual particles was noted.

Silt - 159 PCM

The release of silt at 159 PCM resulted in a fairly even distribution of solids across the tank bottom, but a tendency to a ring formation remained as shown in Figure 3-105. The entire tank bottom had some accumulation of particles indicating that as a result of descent and impact, the dredged material became well dispersed in the water column. The maximum depth of deposit was 0.2 cm at the center of the dispersion pattern. At the edge of the ring the depth was 0.1 cm and outside the ring the deposit was less than 0.1 cm deep at all locations.

Silt - 198 PCM

The silt at 198 PCM spread uniformly over the entire tank so that at all locations the depth of deposit was less than 0.1 cm. This may be seen in Figure 3-106.

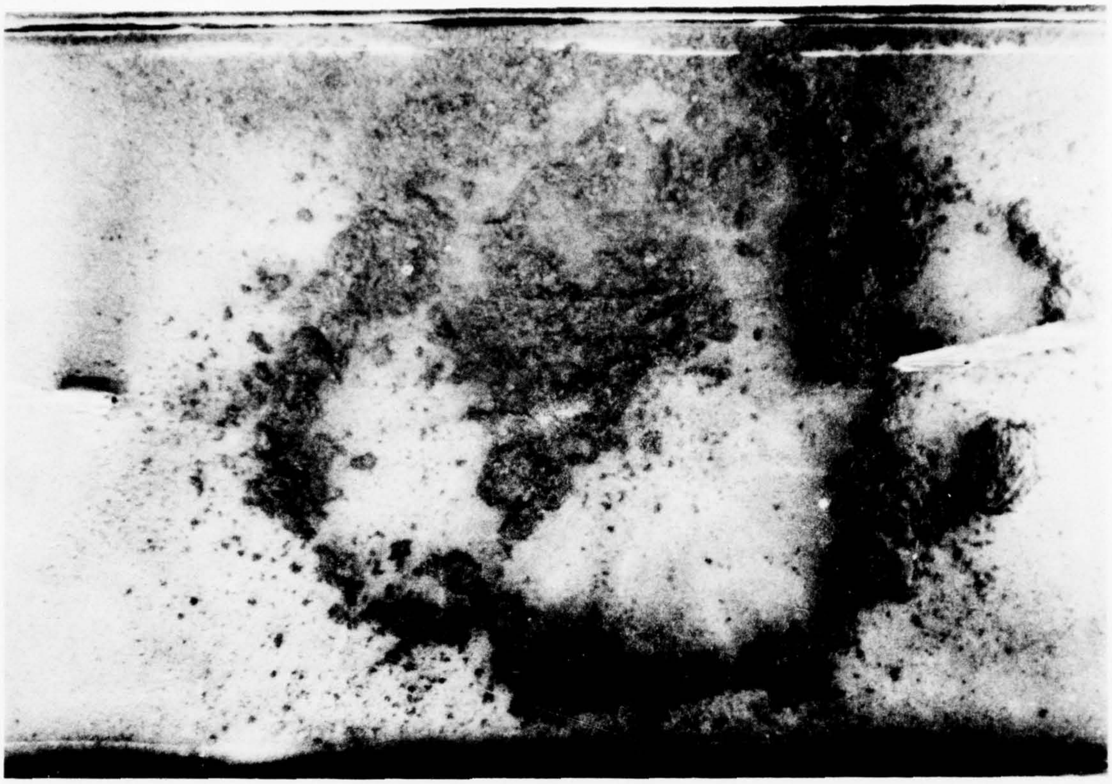


Figure 3-104. Silt 114 PCM

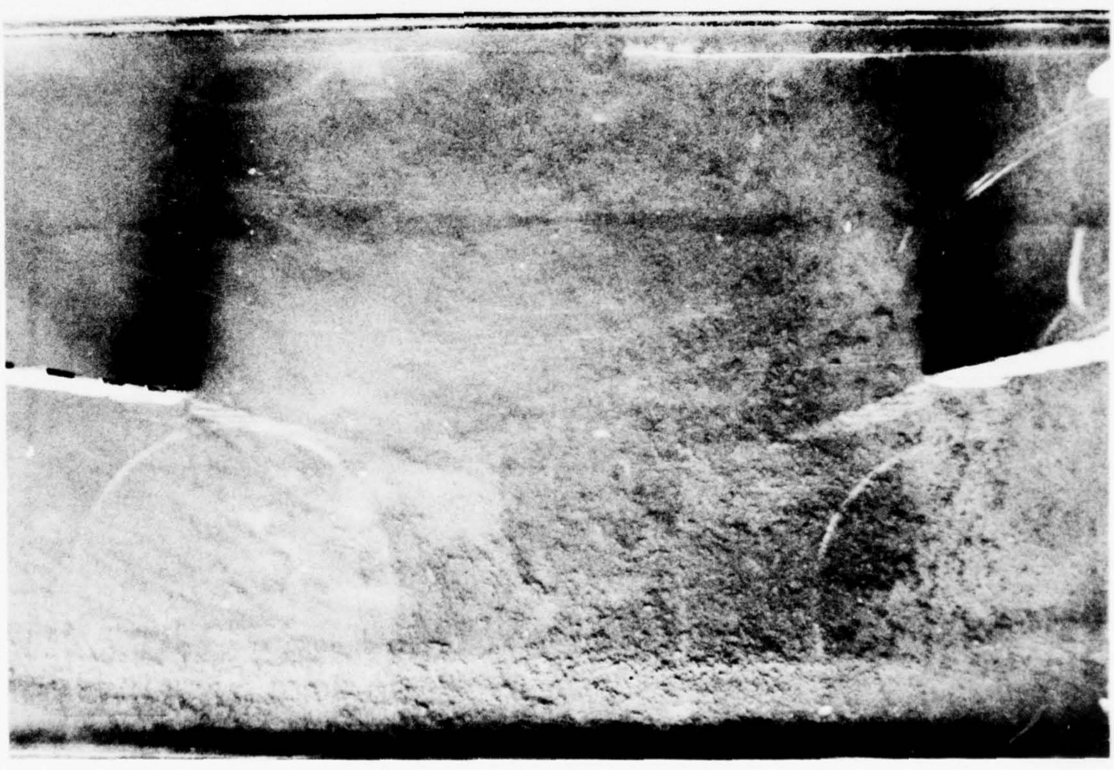


Figure 3-105. Silt 159 PCM

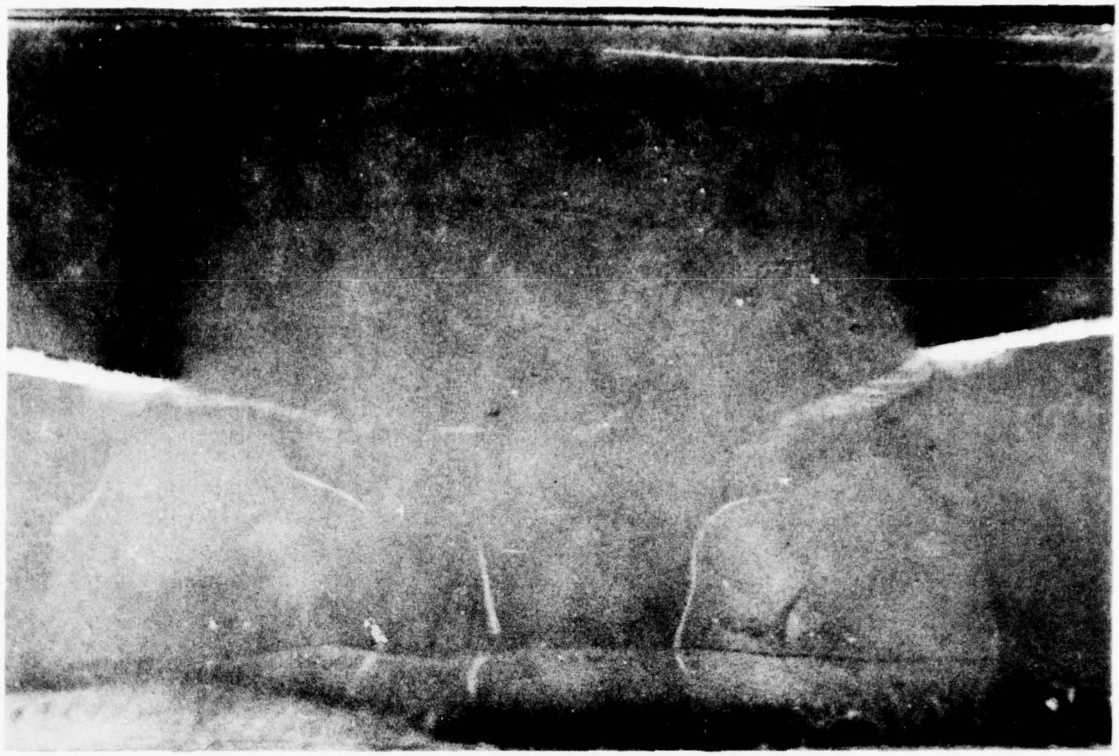


Figure 3- 106. Silt - 198 PCM

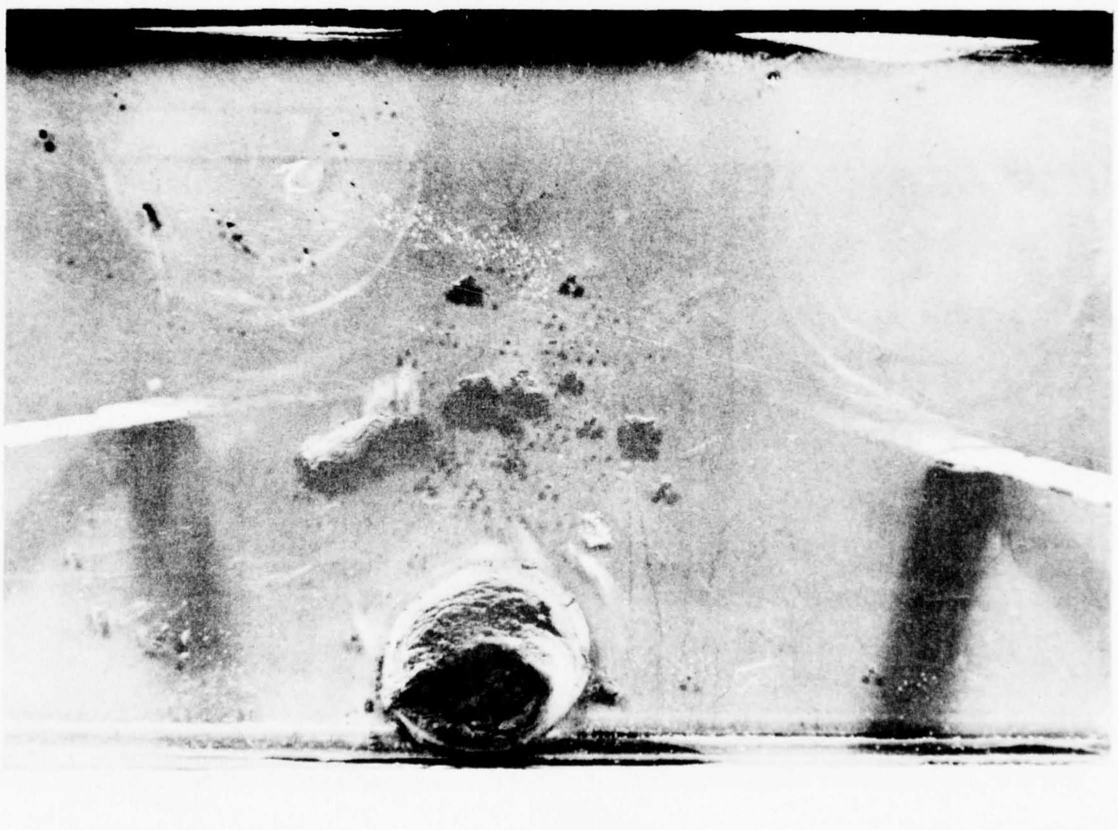


Figure 3-107. Clay 169 PCM

Clay - 169 PCM

The clay at the original percent moisture of 169 PCM fell as one piece except for a small amount which adhered for a short time to the dump container. The main portion fell toward the back of the tank during descent and did not break up on impact although it did become somewhat flattened. The portion which left the dump container a few seconds later fell to the center of the tank and also did not break up on descent or impact but remained near the center. The positions of these lumps are shown in Figure 3-107. The larger portion was about 2.1 cm thick and had a diameter of 8 cm. The largest of the other pieces was 5 cm by 1.5 cm and 0.8 cm thick.

Clay - 198 PCM

Again, as with the 169 PCM dump, most of the material left the container at once, but a small portion fell approximately one second later. The large portion altered its shape to a more rounded configuration as it fell and when it hit the bottom it flattened and slid about 13 cm across the tank bottom without breaking up as shown in Figure 3-108. Its dimensions were then about 13 cm by 8 cm and 1.3 cm thick. The smaller portion remained at the center where it fell and was 8 cm by 5 cm and 0.75 cm thick. A still smaller amount of material spread across the bottom but remained in lumps rather than breaking up to individual particles.

Clay - 211 PCM

Again the clay fell as a single mass and slid across the bottom. Most material was in a portion of the total dump which slid into the wall of the tank after impact. As shown in Figure 3-109, the material remained in large aggregations and did not break down into individual particles. The large portion was about 23 cm long by 4 cm wide and 1.0 cm thick. Other large lumps of material

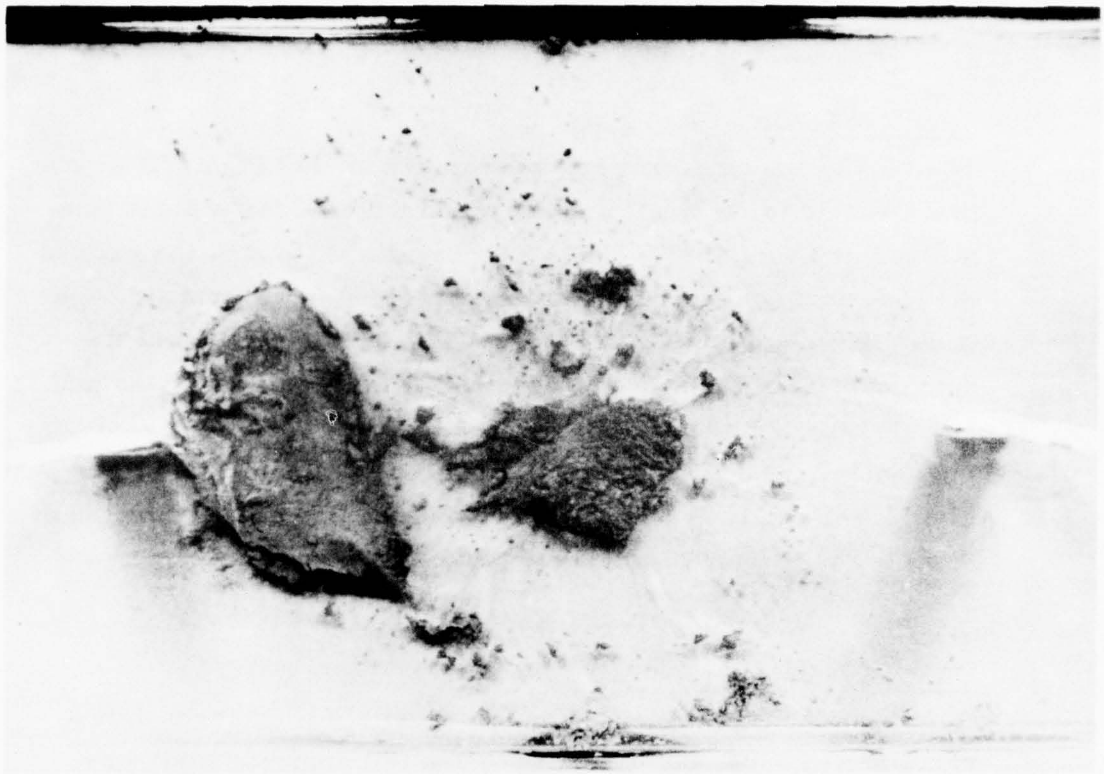


Figure 3-108. Clay 198 PCM

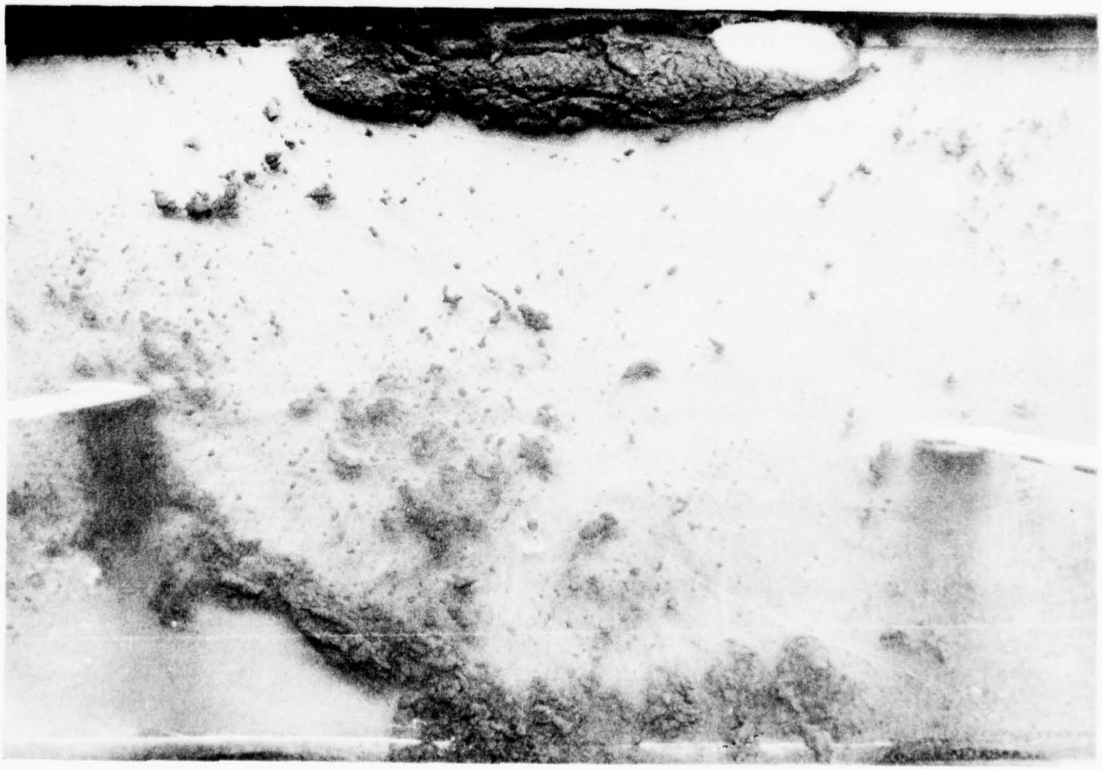


Figure 3-109. Clay 211 PCM

ranged from 0.3 to 0.5 cm thick, but most of the tank bottom contained no deposit.

Clay - 237 PCM

At 237 PCM the clay spread out fairly evenly over the tank bottom. A ring shape was formed as shown in Figure 3-110 with a diameter of approximately 25 cm, but the maximum height of deposit along the ring was only 0.3 cm and averaged 0.2 cm. At the center of the ring the deposit thickness was 0.1 cm and between the center and the ring the thickness decreased to less than 0.1 cm. Outside the ring the thickness was also less than 0.1 cm, but at least a dusting of solids was present throughout the tank.

Clay - 246 PCM

A rapid spread across the bottom occurred when the water content was 246 PCM. As shown in Figure 3-111 there was some tendency to formation of a ring, but the thickest deposit along the ring was only 0.1 cm while at the center the thickness reached 0.2 cm. Solids were deposited throughout the tank to a fairly uniform depth, but clumps could still be seen.

Discussion of Results

The purpose of this series of tests was to determine the effect of the moisture content of dredged material on the degree of mounding. It was found that the water content has a very pronounced effect on dispersion and subsequent mounding. Below a critical moisture content the mass of particles acts as a single unit and settles with very little dispersion.

As the moisture content is increased, the dumped material breaks up in the water column and on impact to form a ring pattern with smaller amounts of material within the ring and still less distributed

Figure 3-110. Clay 237 PCM

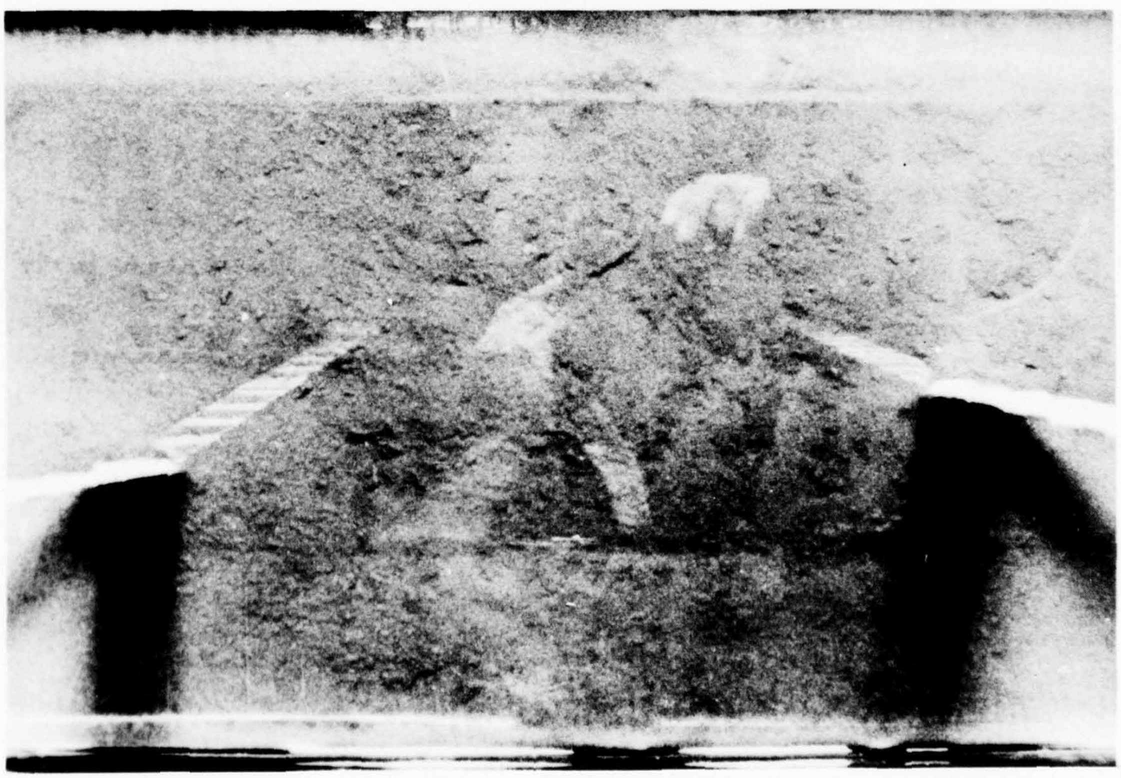


Figure 3-111. Clay 246 PCM

outside. When the moisture content is increased beyond a second critical point the ring pattern is obscured and widespread and generally uniform dispersion results. It appears that in this region rapid entrainment of water into the settling material aids in dispersion. At lower moisture content the individual particles remain in large clumps which greatly limits the degree of entrainment.

A great deal of difference was noted between the mounding characteristics of the silt and the clay. While the results were essentially identical qualitatively, the PCM at which the different types of behavior occurred for the silt and clay were:

| | No Dispersion to Ring-Shaped | Ring-Shaped to Uniform Dispersion |
|---|---|---|
| Silt | 110 PCM | 160 PCM |
| Clay | 220 PCM | 250 PCM |

Figure 3-112 shows the maximum mounding height observed, as a function of percent moisture and percent solids. The silt is seen to mound at a lower percent moisture than the clay and the transition zone appears to be more abrupt.

Figure 3-113 shows the mounding as a function of multiples of the liquid limit of the material. It appears as though the two materials can be characterized as mounding for multiples less than 2 and flowing for multiples greater than about 3.3. Thus, the transition range between total mounding and total dispersion is seen to be small, for the depth and material volumes of these tests.

Exact identification of the transition points may be somewhat uncertain, but it is clear that the behavior of the two materials is different. The silt is seen to have a tendency to dispersion at a

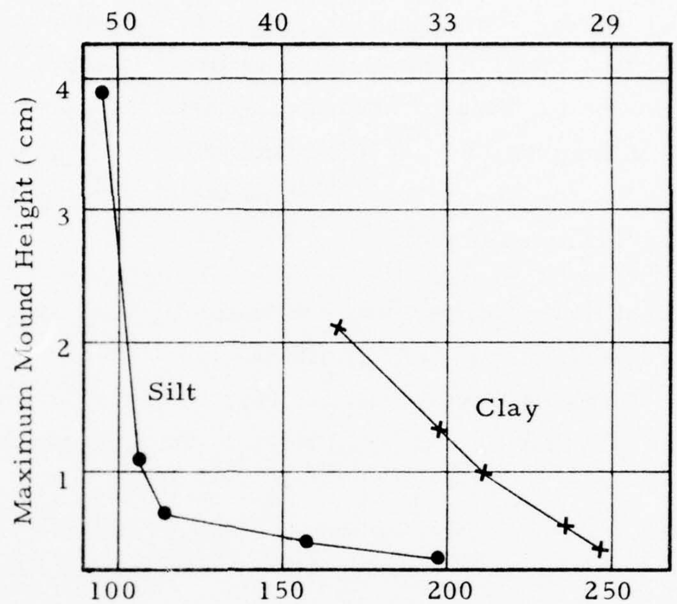



Figure 3-112. Maximum Mound Height as a Function of Percent Moisture

Percent Moisture

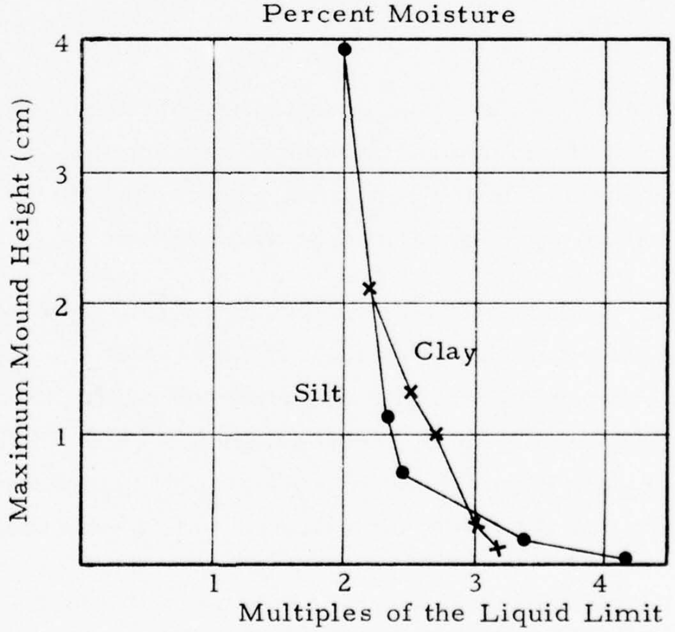



Figure 3-113. Maximum Mound Height as a Function of Multiples of the Liquid Limit

lower water content than the clay. This is probably due to considerably lower cohesive forces between the silt particles as compared to the clay particles.

It should be emphasized that these tests were run in only 16 inches of water and that the same relationships may not hold as the depth is increased.

## F. Conclusions

### 1. Dredging Phase

The literature review, coupled with observations and measurements made during this phase of the study indicate the following:

a. The capability exists to maneuver the dredge anywhere in San Francisco Bay with a navigation accuracy of several feet. However, clamshell and cutterhead dredges have inherently better control for selective removal of small volumes of material. Hopper dredges are more effective when used to dredge a channel, rather than a discrete spot.

b. All three type dredges generate near-field effects, such as turbidity and oxygen depletion in the water column. While the effects are highly variable and depend on the type material being dredged, there is reason to believe that the turbidity plume behind a clamshell dredge and an overflowing hopper dredge is probably higher than behind a cutterhead dredge. For material containing a high percentage of clay-size particles, the entire water column is affected and there seems to be little change with range downstream. For material containing large fractions of silt and sand, there appears to be a decrease in the turbidity in the lower part of the water column as range downstream increases.

c. Oxygen depletion can occur in the water column downstream of any dredge but the effect is probably minimal due to dilution. In the case of a clamshell dredge, elevated oxygen levels have been observed due to the aeration that takes place when the bucket is lifted through the water column. Overflow of low-oxygen water from a barge does not appear to significantly reduce the oxygen levels in the surface water. This is probably true for the weir overflow water from the hopper dredge as well.

d. For values of sediment oxygen demand of interest, there is no significant satisfaction of the demand due to the dredging operation for any of the dredge types used in San Francisco Bay.

e. The moisture content found in situ and in a barge filled by a clamshell dredge in Alameda were approximately the same. There is every indication that clamshell dredges can recover dredged material at approximately the same moisture content as in situ. However, the variability within a barge pocket is quite high and depends on the degree of disturbance that the dredge imparts to the material. Moisture content within the barge pockets is dependent on the way that the barge is filled. If a barge were allowed to overflow as a hopper dredge does, the observed high moisture content near the surface in the end pockets would undoubtedly decrease. In general, the mean moisture content for a total barge load when filled with a clamshell can be quite close to the in situ moisture content.

f. Moisture content in the hopper dredge was found to be low in the lower portions of the hopper and exhibited a steep gradient in the upper portion of the hoppers. In the case of Mare Island, the values in the lower portion of the hoppers approached the in situ value for Mare Island Straits.

g. The hoppers appear to contain two discrete layers. The bottom layer is relatively homogeneous and of low moisture content approaching the _in situ_ value. On top of this layer is a high gradient layer which slowly settles out over time. The dividing line between these two layers can be controlled by controlling the amount of overflow allowed. The longer the dredge operates the deeper the low moisture content layer becomes. The location of this dividing line varies from hopper to hopper and is dependent primarily on the settings of the partitions on the chute inputs which control the distribution of material coming out of the pipeline.

h. Shear strength measurements in the dredged material were highly variable in the clamshell and very low in the hopper dredge. Since the _in situ_ values of moisture content are close to, or above, the liquid limits for the materials measured, the shear strength must be low by definition. There is every indication that the shear strength in material dredged with a pipeline or hopper dredge should be approximately zero. With a clamshell dredge, the values should range from zero to _in situ_, depending upon the way the clamshell obtains and releases the material. Shear strengths generally were under 100 psf. Shear strength does not appear to be a good descriptor for dredged materials due to their relatively high water content. Thixotrophy does not appear to be significant over the time intervals of interest.

i. Penetration and sinking sphere tests did not yield any unexpected results concerning bearing strength of the material. However, the sinking sphere appears to be an excellent analytical tool for hopper dredge use as it can measure the approximate location of the low moisture content interface. Experiments with different density spheres could yield a valuable, yet simple tool for hopper dredge operators to assess the effectiveness of their

dragarm operators, the settings of the chute discharge baffles, and other operational variables.

2. Transport Phase

a. Vibrations appear to have little effect on the moisture content of the material in the hoppers but ship maneuvering appears to cause shifting of the material which releases pockets of water and results in settling of the material. Settling of the high gradient upper volume of the hopper takes place but this effect requires time. Over a period of one hour after dredging ceases, this can add several feet of low moisture content material to that in the bottom of the hopper.

b. Vibrations measured on the HARDING were extremely low. There is a coupling loss between the walls of the hoppers and the dredged material in the hoppers. One of the larger sources of vibrations during the dredging phase is the pump.

c. There is a tendency for the moisture content at a given depth in the hoppers to decrease with time. This is most prevalent in the upper half of the hopper where the moisture content is high.

d. Maneuvering of the vessel caused water trapped in the lower portions of the hopper to be released allowing settlement of the low moisture content layer. It was also noted that the top of this layer shifted significantly as the vessel heeled. Settling might be enhanced by maneuvering the vessel but this effect is probably not significant.

e. Measurements over 24 hours demonstrated that thixotrophy is not an important parameter over the time periods that dredges normally transit to a disposed area. However, the effect was significant in material stored in barrels for 8 weeks.

f. Comparisons of static and vibrated samples demonstrated that vibrations in the frequency range from 0.2 to 60 hertz have virtually no effect on the settling of the dredged material. There was an indication that for high moisture contents the settling rate is actually decreased during vibration, but this effect is probably not significant.

g. For the samples tested, shear strength was extremely low and did not increase with vibrations.

h. It can be concluded that vibrations such as those measured on the HARDING do not have a significant effect on the physical characteristics of dredged materials. However, motions induced by maneuvering, and possibly by vessel motions in a sea way, do cause a decrease in the average moisture content in the lower portions of the hopper.

3. Disposal Phase

a. There is very little data available in the literature which can be used to quantify the hypothesized behavior of dredged material when disposed of in open water.

b. Existing mathematical models, such as the Koh Chang and the Kristnappan models, do not allow adequate description of the dredged material physical characteristics. When these models are used to predict the dynamics of the scaled tests conducted on this study the results are unsatisfactory. Neither model provides the capability of inserting moisture content as a parameter whereas the observed mounding and bottom flow is highly dependent upon this parameter.

c. For many of the cases of interest, involving low to medium moisture contents, scaling is feasible using the Froude number.

This is a valid method for cases where inertial effects predominate and the dump behaves as a cloud rather than as individual particles. Froude number scaling appears to be appropriate, for the cases of interest, throughout the descent phase and over a portion of the bottom flow phase. When in the region where diffusion is predominant, Froude number scaling does not appear to be applicable.

d. The descent velocity shows three distinct phases; an acceleration phase, an equilibrium velocity phase, and a deceleration phase. In general, the acceleration phase is very short, perhaps reaching the equilibrium velocity before the material leaves the dump container. In some cases the clay material appeared to reach the deceleration phase before the material left the container. For some material characteristics (i.e., low moisture content) the equilibrium velocity was maintained until impact and there was no deceleration phase.

e. The collapse phase and bottom flow phase appeared to be highly dependent on the material characteristics. Low moisture content resulted in mounding directly under the dump point, little horizontal transport of material, and low bottom flow velocities. High moisture content was just the opposite. There appears to be a transition zone between these two extremes and its size appears to be related to the material physical characteristics, the dump volume, and the depth in which the dump takes place.

f. For the dump vessels simulated in this study (barge and hopper dredge) there appears to be little change in the ultimate dispersion pattern achieved. A number of results tend to indicate that for the cases of interest the size and shape of the dump vessel's opening is of secondary importance and that the more important parameter is the volume of the material that is dumped. The

size and shape of the opening appear to have an affect on the descent characteristics for a short period of time while leaving the vessel. After that the remainder of the descent phase appears to be relatively independent of the dump container characteristics.

g. Dumping in fresh water or salt water appears to have little effect on the dispersion achieved. The descent velocity appeared to be somewhat higher for the salt water drop of clay material than it was for fresh water. In the case of low moisture silt, the descent velocity was also higher in salt water and the material did not appear to break up on descent as easily as it did in fresh water. Other than for a slightly higher descent velocity, however, we feel that the primary effect due to water type (if any) would be flocculation in the turbidity cloud

h. The size (volume) of the dump and the depth of the water that it takes place in have a significant effect upon the dispersion pattern. Volume and depth are intimately related and may be scaled under many conditions of interest using the Froude number. While the effects are also related to the moisture content of the material, it appears as though descent velocities increase and entrainment decreases with increasing dump volume. The average bottom flow velocity increases with increasing dump volume, as does the percentage of material in the vicinity of the impact point. For the materials tested (PCM 190) the entrainment coefficient is independent of dump volume. For a given dump volume the descent profile appears to be independent of depth thus the characteristics observed on a shallow dump may be extrapolated to a deeper dump. However, this is not true in the first two feet and future tests should have a minimum depth of four feet. For a given volume, dumping in shallow water results in a smaller dispersion pattern on the bottom than is achieved when dumping in deeper water. Scaling the results of the

four foot and nine foot drops verified that Froude number scaling is valid for some parameters of interest.

i. Moisture content appears to be the predominant characteristic affecting the dispersion achieved when bottom dumping in open water. There appear to be three ranges of PCM of importance; the "solid phase," the "transitional" phase, and the "liquid" phase. For materials with PCM's in the solid range, dumps are characterized by a very rapid descent phase, little cloud growth and little spreading of the material on impact. The material falls as a block, at equilibrium velocity, and entrainment is negligible. Most of the material mounds directly under the dump point.

The liquid phase is characterized by a slower descent phase, the cloud expanding due to entrainment, and a rapid flow of material across the bottom after impact. There is little, or no mounding.

The transition phase is characterized by conditions that vary from the solid to the liquid phase. This phase is variable in width (PCM) and the upper bound (the beginning of the liquid phase) is a function of depth and volume as well as material type. Materials that start their descent with PCM values in the "transition" phase may convert to the "liquid" phase while descending, due to entrainment of water (hence increasing their PCM). We are hypothesizing this behavior of the transitional phase, based upon several tests in the large tanks and the aquariums.

There is some indication that cohesive materials flow less (mounds more) than non-cohesive materials and this may be due to the vertical momentum being dissipated on impact by having to overcome the cohesive forces. If this is true, dumps from a barge filled by a clamshell dredge might experience less dispersion than dumps from a hopper dredge, for the same moisture content.

There appears to be a relationship between the liquid limit of materials and their position in the solid, transition, and liquid phases mentioned above. Preliminary tests show that materials with a percent moisture less than twice their liquid limit are in the solid phase and above 3.3 times their liquid limit they are in the liquid phase. These tests were conducted in a small aquarium and the 3.3 multiple will probably change with volume and depth. However, it is hypothesized that the bound for the solid phase (2 x LL) will remain the same.

## G. Recommendations

### 1. Dredging Phase

The dredging operation modifies sediment physical characteristics and there is an observed change in percent moisture, percent solids, dry density, wet density, shear strength, bearing strength, etc. It is not clear which characteristic best describes the changes due to the dredging operation that are most important to the behavior of the material during dumping. The material appears to fall into a region that is interdisciplinary (between classical soil and fluid mechanics). Studies should be conducted to establish which physical characteristics are the most important. Once these characteristics are identified, field measurement should be carried out to determine the values *in situ* and as a function of dredge type. This data may then be used to predict the behavior of the material when dumped in open water.

### 2. Transport Phase

Settling, and the voiding of pockets of water appear to take place as the hopper dredge turns and maneuvers. It is recommended that a monitoring program be started, using sinking spheres to establish the magnitude of this effect and to determine whether or not it is possible to significantly increase the dredge payload via

maneuvering and then adding more dredged material. In cases where the moisture content is just slightly in excess of the maximum value for which mounding will occur, this activity could possibly decrease the moisture content sufficiently to cause mounding. Sinking sphere measurements could also yield valuable information on how to maximize the performance of the dragarm operators and optimize the settings of the chute baffles.

A program utilizing sinking sphere tests, supplemented by samples obtained using the relatively simple syringe sampler would substantially increase the understanding of the importance of settlement time and maneuvering on the behavior of dredged material. It would also yield valuable input information that could be used to estimate the behavior of material after it is dumped in open waters.

3. Disposal Phase

The small and large test tank investigations demonstrated that the physical characteristics of dredged material are highly significant as related to the dispersion of the material when bottom dumped. There appears to be a significant difference in parameters such as descent and impact velocity, cloud radius, mounding, and bottom flow as a function of moisture content. These differences should be investigated further to define the controlling characteristics and to quantify their effects more precisely. The importance of cohesiveness should be investigated further. The relationship between liquid limits and material type should be further investigated for a number of different type dredged materials. Full scale tests should be conducted to verify the effects seen in the laboratory and to establish the feasibility of Froude number scaling as an alternative to sophisticated mathematical modelling. Once the controlling physical characteristics are identified, and the scaling laws verified, it will be possible to predict the descent,

impact, mounding, and flow behavior for material dredged from any part of the Bay. This will be a valuable input for environmental impact statements and assist in establishing the short and long term behavior of material when bottom dumped in open water.

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## IV. EVALUATION OF INTERTIDAL DISPOSAL

The San Francisco District in recent years has been planning, researching, and beginning to implement, the creation of tidal wetlands using dredged material as a base. This program can achieve two worthwhile goals: restoration of some of the large area of wetland which has been lost by diking and filling, and disposal of dredged material in such a way as to be environmentally beneficial.

Early studies by the District have emphasized the biological aspects of marsh building [1, 2]. It has been pointed out by Kerr and Cali [3] that many of the available studies of marsh building have been on sandy substrates, and that "little is known about artificial marsh establishment and growth on organic, silt, and clay substrates." Most of the dredgings from San Francisco Bay are clays and silts. Another stimulus for the biological emphasis in the District's studies is the uniqueness of marsh plants indigenous to the Bay Area. The most important of these are *Spartina foliosa* and *Salicornia pacifica*. Much of the available literature on marsh establishment has resulted from work on the Atlantic and Gulf Coasts, where the plants of interest are typically *Spartina alterniflora* and, at higher elevations, *Spartina patens*.

To complement the biological studies of germination, growth, and transplanting, there is a need to evaluate certain specific questions related to physical and engineering considerations. These items for study, identified by the District for this contract, are discussed in this chapter:

A) Controlling fill elevations

B) Methods of opening the area to tidal action (all proposed marsh building sites are behind existing dikes)

C) Intertidal stabilization of sediments

D) Laboratory studies to evaluate movement of salt from the beds of abandoned evaporation ponds through dredged material.

A. Controlling Fill Elevations

1. Need for Control

Elevation

There are a very few species of plants which dominate the existing salt marshes of San Francisco Bay. Since these marshes are highly productive and offer a suitable habitat to many desirable faunal groups, it is logical for those who would build new marshes to attempt the establishment of those plants found in existing marshes. Those plants dominate the marshes largely because of the cyclic flooding and ebbing of tidal waters. Each plant (for example, Salicornia pacifica and Spartina foliosa) favors a general zone of elevation which can be characterized by an approximate ratio of time submerged to time exposed. Accordingly, to establish these plant communities, the substrate must begin and remain at an elevation at which these plants can thrive.

The marshes of the Bay Area have been studied by many investigators with respect to the relationship of plant groups to elevation in the intertidal zone. Pestrong[4] described tide ranges, presenting the data repeated herein as Table 4-1. Another report [1] discussed the elevations at which various desirable marsh plants are commonly found. That information is expressed schematically in this report as Figure 4-1. On the same Figure are shown the proposed elevations for marsh building near Alameda Creek from an Environmental Impact Statement published by the District [5] .

Tolerable errors

It is expected that all material placed with marsh creation in mind will be from hydraulic dredges. Given the undeveloped state of the art of predicting the settlement of a slurry from a hydraulic dredge, it is helpful to consider whether a fill elevation which is ultimately "too high" is preferable to one which is "too low". Several aspects of the individual site must be known in order to

Table 4-1. Tidal Elevations and Ranges in San Francisco Bay Given in Feet, As Reported by Pestrong [4][(1)]

| | Mean higher high water feet | Mean lower low water feet | Mean range feet | Estimated maximum high water feet | Estimated minimum low water feet | Maximum range feet |
|---|---|---|---|---|---|---|
| Golden Gate | 2.6 | -3.1 | 5.7 | 5.1 | -5.6 | 10.7 |
| Point San Pablo | 2.9 | -3.0 | 5.9 | 5.6 | -5.4 | 11.0 |
| Selby | 3.2 | -3.1 | 6.3 | 5.9 | -5.1 | 11.0 |
| Benicia | 3.3 | -2.5 | 5.8 | 6.0 | -4.5 | 10.5 |
| Confluence of Sacramento and San Joaquin Rivers | 3.3 | -1.1 | 4.4 | 6.0 | -2.5 | 8.5 |
| Hunters Point | 3.3 | -3.3 | 6.6 | 5.7 | -5.6 | 11.3 |
| San Mateo Bridge | 3.5 | -3.8 | 7.3 | 6.2 | -6.3 | 12.5 |
| Dumbarton Bridge | 4.1 | -4.3 | 8.4 | 6.8 | -6.7 | 13.5 |
| Alviso (south end of bay) | 4.4 | -4.6 | 9.0 | 6.9 | -7.1 | 14.0 |

(1) All elevations in SLD - Sea Level Datum

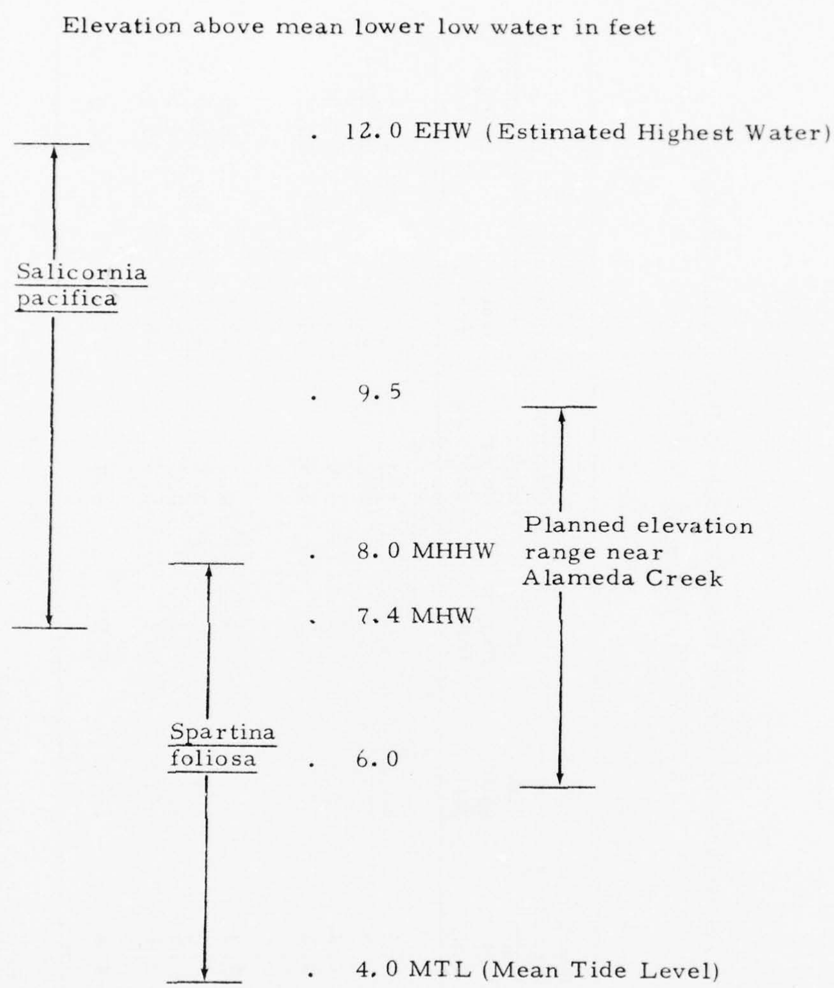


Figure 4-1. Schematic Representation of Marsh Plant Elevation Ranges in San Francisco Bay [1]

answer this question. Frequency of dredging in the local area should be considered. If the marsh is to be built near a channel requiring frequent maintenance dredging, a lower than optimum elevation is preferable to a high elevation. After the finding of unexpectedly large settlement, the area can be filled to the desired elevation during a subsequent maintenance dredging operation. Conversely, if future dredging in the area is not expected to be frequent, a higher than optimum elevation is probably a preferable error. In this event, the material can be worked after drying to form a combination of marsh and small hills or ridges, which will provide some terrestrial habitat and an opportunity for access to the marsh.

The surrounding ecosystems should also be evaluated when considering whether an error on the low side or the high side is preferable. If the fill site is near large areas of high marsh, perhaps a mud flat is a more biologically valuable addition. Such an area caused by an error on the low side, if exposed to a source of detrius and if in an area of accretion, can be expected gradually to evolve into a marsh through natural processes.

Topography

Not only the range of absolute fill elevations, but also the topography is important in marsh building. Slopes should be gentle enough to avoid rapid and turbulent water flow which could cause erosion. Steep slopes would also favor rapid drainage which is undesirable because marsh plants require waterlogged soils. In addition, a general slope rising to the landward end of the marsh is desirable. If low areas were present at the landward side of the marsh, they would not drain properly. The consequence would be ponding, solar evaporation, and the development of high salinity pools which would inhibit plant growth.

2. Control Methods

Basic and Applied Soil Science Considerations

Certain fundamental concepts of soil science should be the basis for evaluating the engineering methods for controlling fill elevations. The important parameters of the dredged material should be defined and characterized so that soil behavior as affected by those parameters can be predicted. The response of the important parameters should in turn be related to various possible engineering methods. Unfortunately, much of the work done in the past in evaluating hydraulic fill behavior is not directly applicable to the problem at hand. The reasons for this situation are primarily:

(1) Hydraulic fill materials have been selected, where possible, to be competent foundation materials unlike the silts and clays dredged from navigation channels in San Francisco Bay (Bay mud).

(2) Where fill materials have been similar to Bay mud, investigations have focused on the materials' ability to support loads. Engineering techniques have been investigated for enhancing load-carrying ability, but little work has been done in evaluating the natural and unaided consolidation and settlement of dredged materials. Accordingly, there is little in the literature to aid in predicting fill elevations after consolidation; a study to advance the state of knowledge in this specific area was recently begun at the Massachusetts Institute of Technology (U.S. Army Engineer Waterways Experiment Station, Dredged Material Research Program, Work Unit 4A16).

The literature does contain some theoretical treatments of dredged material consolidation, and some insight may

also be gained from a report dealing with prospects for reclaiming submerged San Francisco Bay lands for agriculture [6 ] . The referenced agricultural study related literature information and the results of some laboratory experiments to describe how sediments behave upon drying. The most important general statement which can be made based on that report is that the loss in volume which accompanies dewatering is basically irreversible. Rewetting does not restore the material to its initial volume.

Tests were performed on drying and rewetting of San Pablo Bay sediments in the referenced study. Cylindrical samples of cored sediments shrank in length by 24% and in diameter by 18% after 18 days of air drying. Upon re-saturation, the increase in volume was only sufficient to reduce these dimensional losses to 21% and 16%, respectively [6] . These investigators compared their data to that developed by Dutch researchers, and both studies indicated that high clay content increased the sediment's tendency to shrink. This shrinkage is probably related to the collapse of loose, flocculated clay particles upon water removal.

Other more recent investigations have considered empirical observations of the changes in volume of dredged material from its <u>in situ</u> condition in the channel, through the slurried phases of hydraulic dredging and transit in hopper dredges, to the reduction in volume which accompanies dewatering behind a dike. Johnson [ 7 ] and Kolessar [ 8] discussed Bulking Factors (B. F.). The bulking factor is loosely defined as the ratio of dredged material volume after hydraulic dredging and drainage to the same material's prior <u>in situ</u> volume. The looseness in the definition arises from the lack of a standardized time for

consolidation. Since all hydraulically dredged materials experience some time-dependent reduction in volume after deposition within a diked area, a bulking factor based on long-term volume measurement (e.g. several years) will be smaller than one based on near-term measurement. Most bulking factors reported for silts and clays range from 1.4 to 2.0, and these literature values refer to the short-term volume (on the order of one year), since the main use of the B.F. is to compute the volume required to hold a known *in situ* volume of material to be dredged on a single project.

For the purpose of estimating and controlling fill elevations, such empirical and approximate parameters as the B.F. are of limited use. Of far more interest is the volume-time relationship; we wish to know what factors govern that relationship, and the extent to which those factors can be controlled. The ability to make accurate estimates of this relationship is limited by the difficulty of predicting the time rate of change in void ratio of dredged material during consolidation in a diked area.

Considerable research has been done on the behavior of dredgings in diked disposal areas in the Great Lakes by Krizek and his colleagues at Northwestern University and the University of Wisconsin. One paper resulting from work by Krizek and Giger [9] reported on the monitoring of elevation changes within several confined areas near Toledo, Ohio. These confined disposal facilities were surveyed over a nine-year period. In addition to topographic surveys, borings were taken, and dry densities were measured. The resulting data led to development of an approximate formula, which the authors felt was valid for a period of at least

ten years:

$$V_D = 0.8\,(1-0.04t)\;V_B \qquad (1)$$

where:

$V_D$ = volume of dredgings in the disposal site after t years of consolidation

$V_B$ = in situ bottom sediment volume (pre-dredging)

The linear volume reduction of 4% per year shown in Equation (1) will of course not continue indefinitely, as some limiting density would ultimately be attained. Some variation in this 4% value is also likely; surveys of one area revealed a compaction of 6% of the initial dredged material thickness in the first nine months.

A further word of caution is in order regarding the coefficient value of 0.8. This factor was calculated by Krizek and Giger as the ratio of dredged material volume in the disposal area, after settling and decanting the supernatant (for a year or two), to the pre-dredging in situ volume. Clearly, this factor will be highly sensitive to local conditions and operating procedures. The initial dry density of material discharged in these studies was 50 pounds per cubic foot.

Another problem in applying these data to the situation in San Francisco Bay is the Bay area's seasonal rainfall patterns, which contrast to the more equalized monthly precipitation on the Great Lakes, Atlantic, and Gulf Coasts. Of the Bay Area's 24" mean annual precipitation, 80% normally falls between November and April. As a result, during the dry summer, dredged materials behind dikes undergo extensive water loss by evaporation. Vertical cracks result, forming "pillars" of dredged material separated by gaps of one to three or four inches. The winter rains then erode the walls of these "pillars" with the result that a new slurry is formed within the vertical cracks. (Fig. 4-2).

Figure 4-2. Dredged Material "Pillars" at Pond 3, Hayward, Showing Slurry in Cracks after Winter Rains (Photo Taken April 9, 1975)

The effect of this process on the consolidation behavior of the dredgings is uncertain, but the process may lessen the applicability of theories and data generated elsewhere to the volume-time relationships of confined dredgings in the Bay Area.

The information gained from the above empirical studies may be supplemented by other work which considers soil science theory as applied to the settlement of dredged materials. Krizek, Karadi, and Hummel [10] combined field and laboratory work with literature review and theoretical discussions to evaluate various engineering properties of dredgings. As these authors pointed out, the application of classical consolidation theory to the behavior over time of a dredged material slurry produces only a "first order approximation." Two of the primary assumptions of the classical theory are violated: small strains, and complete saturation at all times. One potential problem with the saturation assumption is that some dredged materials will generate gases during consolidation. These gases may eventually escape, but their presence makes the description of volume-time relationships very difficult. Unfortunately, much of the emphasis in the referenced study was on load-carrying capacity of thick deposits of dredgings (on the order of 20 ft.) As a result, the data are generally not applicable to predicting the settlement of thin lifts of dredgings with no superimposed loading.

Perhaps the most applicable study for the purpose of controlling fill elevations is that of Johnson [7], who reviewed the literature and the theoretical basis for sizing containment areas for dredged materials. Johnson presents several theoretical approaches, and concludes that field and laboratory tests on dredged material are required to validate theories and confirm the existing procedures for sizing confined dredged material disposal areas. Using some of Johnson's reported equations, some calculations were made for this study to provide approximations to settlements which might be expected in marsh

creation efforts in the Bay Area. These equations are based upon field observations of dry density variation with time for alternately submergent and emergent sediments:

$$\text{For clays:} \quad \gamma_d^t = 46 + 10.7 \log_{10} t \tag{2}$$

$$\text{For silts:} \quad \gamma_d^t = 74 + 2.7 \log_{10} t \tag{3}$$

where: $\gamma_d^t$ = dry density at time t, $lb/ft^3$

$t$ = time, years

The dry densities were converted to more useful units with the following relationship, also from Johnson [7 ] :

$$S = H \frac{\gamma_d^t - \gamma_d^1}{\gamma_d^t} \tag{4}$$

where: $S$ = settlement, ft

$H$ = depth of sediment, ft

$\gamma_d^t$ = dry density at time t years, $lb/ft^3$

$\gamma_d^1$ = dry density after one year, $lb/ft^3$

When t = 1, S = O from the above equations. Hence, these equations have practical value only for times greater than perhaps two years. The results of calculations are plotted on Figure 4-3, which shows clays to have considerably more settlement than silts, especially for thick deposits of dredged material. Within the limitations of the above equations, it appears that clays may undergo settlements of greater than 20% of the original dredged material thickness in the diked area.

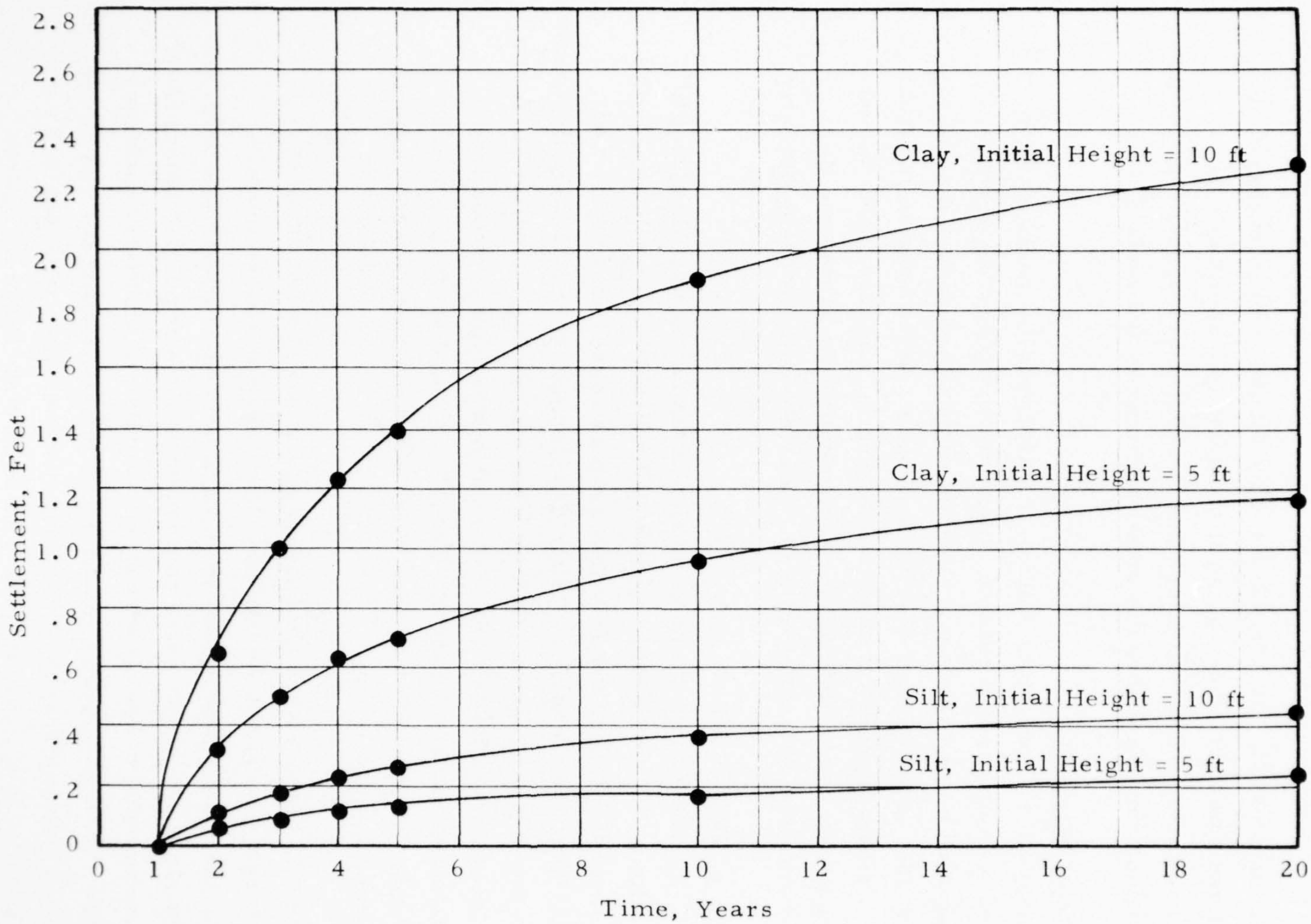


Figure 4-3. Calculated Settlements as Affected by Time, Material, and Thickness of Fill

The equations (2), (3), and (4) cited by Johnson were developed by Lane and Koelzer, whose generalized formulas applicable to several grain size distributions and several submergence/emergence conditions were discussed in detail by Koelzer and Lara [11] .

Material Selection

If economic and practical options are available, a marsh creation project can be carried out with improved control of fill elevation by filling with more coarsely grained dredged materials such as silts and sands. As Figure 4-3 indicates, silts may be expected to undergo only a small degree of settlement with time, and this trend is even stronger for sandy materials. Accordingly, the elevation after deposition and dewatering of such materials will not change greatly.

Means of Deposition

Selecting a discharge mode is an important factor in controlling fill elevations. The discharge from a hydraulic dredge typically results in a mound of large, dense solids such as rocks, clay balls, and debris near the discharge. After dewatering, on a path radiating outward from the discharge point, will be found increasingly smaller particles and increasingly flat slopes. This pattern is typical of confined disposal areas, and exists at "Pond 3", a diked enclosure next to Alameda Creek which has been filled with dredgings for the purpose of establishing a marsh.

Given this trend toward decreasing particle sizes away from the discharge, and given the tendency for clays to undergo more consolidation and settlement than more coarse-grained materials, one might logically expect more settlement in time at points remote from the former discharge pipe location. Control of discharge locations is therefore a potentially valuable tool for controlling fill elevations. Where a flat topography is desired, the discharge pipe should be moved quite frequently to assure an even distribution of coarse

materials. Two methods may be used to avoid depositing coarse materials only near the perimeter of the dike, resulting in lower, clay-rich areas in the center: use of narrow areas where coarse materials in the discharge can reach the center, or extension of the discharge pipe into the center of the disposal area during a portion of the filling period.

Equipment to perform either of these tasks has been considered in a recent report to the Waterways Experiment Station [12] . Spillbarges, swivel pipe joints, branching pipes with Y-fittings and valves, and "whooping cranes" (land-based vehicles on which the discharge pipe is supported) have all been used on dredging projects in this country and overseas.

Operations on Deposited Material

Given the uncertainties in predicting fill settlement over time, it is useful to consider means for increasing the rate of dewatering and settlement. In this way, the fill can be actively compressed in a short time, thus lessening the time lag before passive settlement shows whether the proper elevation has been attained. Corrective measures such as adding more material or excavating the fill can then be implemented at an early date. Methods for increasing the rates of dewatering and consolidation are described in Section V of this report.

Means of compacting fills in diked disposal areas have also been discussed in Appendix J of the San Francisco District's Dredge Disposal Study. Bulldozers, towed multiple discs, and towed sheepsfoot rollers have been used with success at various sites in the Bay Area [13] .

3. Summary and Recommendations

The tools for predicting settlement based on dredged material parameters are not well developed. Current research at the Massachusetts

Institute of Technology (DMRP Work Unit 4A16) should produce valuable information in this regard, perhaps in early 1976.

Pending the availability of that information, approximate predictions of settlement can be made for various materials to be dredged, and initial elevations can be governed by those predictions, with the methods described in the above sections.

Settlement can be hastened, and final elevations can be determined early, by use of compacting equipment such as has been used on Bay Area land disposal sites in the past.

B. Opening the Area to Tidal Action

1. Scope

When a diked area has been filled for the purpose of marsh creation, there often will be several technical problems in opening the area to tidal action. Those problems considered in this section are:

- What should be the size of the dike breach?
- What methods and equipment can be used to provide an adequate inlet when the bayward side of the dike has several hundred yards of existing marsh and shallow water?
- Where should the material excavated in creating the inlet be deposited?

2. Tidal Inlet Design

The question of appropriate size of the tidal inlet to a marsh can be placed in perspective by considering the consequences of errors. If the inlet is not sufficiently large in cross section, flood currents may be unable to carry enough water into the marsh in a short enough time to provide the desired time of submergence for marsh plants. If the inlet is

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too large, tidal currents within the inlet will be sluggish and accretion will probably result, until an equilibrium flow area is achieved. At this equilibrium flow area, tidal currents will attain a natural self-cleaning velocity for the materials on the bottom. It would therefore appear that a very large inlet is the preferable error. On the other hand, consideration of the excavation cost and the amount of overburden to be disposed of indicates that the channel should be made just large enough to maintain a self-cleaning tidal current velocity from the beginning.

A review of the literature has led to some relevant facts in determining proper inlet areas and velocities:

- The equilibrium flow area of an inlet depends to a minor extent, if at all, on the grain size distribution of the bottom materials [14] .
- The mean maximum velocity for a wide variety of tidal inlets in America and in Europe is remarkably consistent among locations: approximately 1 meter per second [15] .
- Velocity averaged throughout the tidal cycle in a wide variety of tidal inlets has a wider range, but is still fairly consistent among locations: 0.54 to 0.97 meters per second among 11 inlets on the Pacific Coast, Gulf Coast, and Europe [15].

These observations indicate that the inlet should be designed to produce these velocities according to the continuity relationship $Q = vA$, where

$Q$ = Tidal prism of the marsh (calculated from elevation and topography) divided by time from slack to slack ($\sim 1/2$ of tidal cycle)

v = Design average velocity (say 0.6 meters/sec)

A = Inlet area

The successful breaching of the dike at Faber Tract in South San Francisco Bay and subsequent marsh development offer a means to verify this approach. If current measurements in the inlet were in agreement with the above cited values, one could gain confidence that the literature applies directly to this question. The cause of any large differences is likely to be the fact that the above-cited data are from rather large tidal inlets.

3. Inlet Excavation Methods

Where marshland and shallow water are present between the dike to be breached and the open Bay, difficulty can be expected in excavating the inlet through the marshes and shallows. Access from land will be forestalled by the weak marsh soils. Access from the water by a dredge or other barge-mounted excavation equipment will be hindered by lack of available draft.

The most promising method for overcoming this obstacle is the use of a dragline with a "deadman" (Figure 4-4). The deadman could be mounted as shown on the dike, permitting use of a barge-mounted dragline, or it could be mounted on a barge or on a piling in the water, permitting use of a land-based dragline. This technique has been successful with bucket capacities of 3, 6, and 10 yards and over distances up to 1500 feet [12]. Standard equipment is available in the form of crane-operated scrapers with a deadman, manufactured by Sauerman Bros., Inc. of Bellwood, Ill. [16]. Such equipment has been used to remove sediment from settling ponds at construction sites.

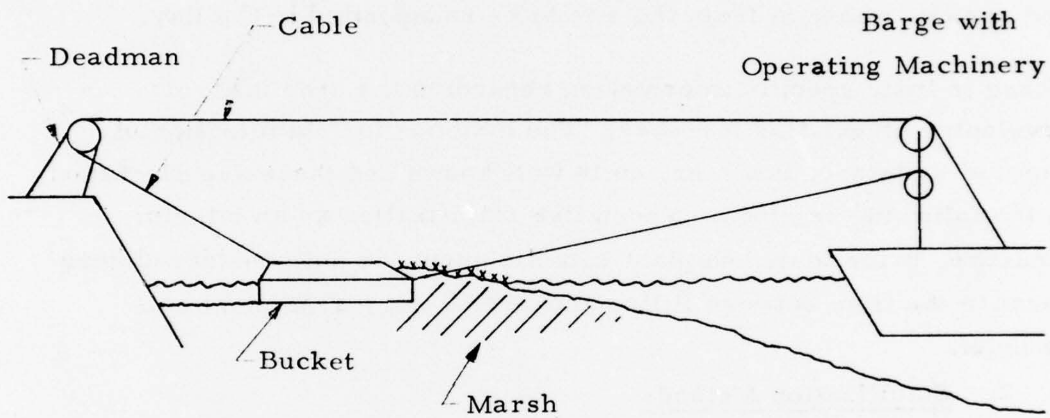


Figure 4-4. Possible Method for Excavating In Inaccessible Areas

4. Overburden Disposal

With the operation shown in Figure 4-4 excavated materials could be placed on a barge and dumped in water at an approved dredged material disposal site. Alternatively, material could be placed within the diked area at locations where elevation should be raised, or they could be deposited at any locations on the dike which require reinforcement.

## C. Intertidal Stabilization

### 1. Need for Stabilization

Erosion is a potential problem whenever bare soils are exposed to moving water. Unconsolidated, fine-grained Bay mud dredged from navigation channels must be considered inherently erodible; erosion is the initial process which ultimately led to redeposition of this material in the navigation channel. Erosion, through the action of rain or of tidal action, has the potential to level slopes, form unwanted gullies, redeposit sediment in undesirable places, and remove material from the site to be redeposited in the Bay.

There is little specific information regarding the erodibility of developing or existing marshes. The methods for stabilization of exposed soils are, however, quite well known and these are discussed in the following sections. Vegetative stabilization as an interim measure, prior to marsh plant establishment, is not considered here because the time between filling and setting out marsh plant will be brief.

### 2. Stabilization Methods

#### Physical Methods

Various types of mulches and mats are physical means of stabilizing soil against erosion. These techniques are very useful for the avoidance of rainwater erosion at construction sites, but less applicable in general for the purpose of stabilizing dredged material while it is being re-wetted by tidal action. Most mulches will tend to float and be washed away, while soil mats would prevent the wetting of the dredged material. Some type of close-mesh net structure might be useful, however. As erosion and sediment control have become more publicized in recent years, several such products have been developed. Laws and ordinances in areas such as Maryland and Southern California, together with the efforts of the U.S. Department of Agriculture's Soil Conservation Service, have increased the use

of meshed blankets and nets at construction sites. Blankets of excelsior or fiberglass, and nets of jute or plastic may be useful. Many of these products have accompanying anchoring systems which were developed for wind resistance, but which should resist the forces of tidal action as well.

Chemical Methods

A few chemical products are available which have been developed to bind soil for erosion resistance. Some of these bind soil by themselves and are often sprayed with grass seed at construction sites to resist erosion while the seed germinates. Other chemical products have been developed for use as "mulch tacks", or materials which bind mulch to soil so that wind and rain do not remove it. "Mulch tack" products have not been developed for uses where total submergence occurs, so their success in making a conventional particulate mulch useful for the problem at hand is doubtful.

No adverse environmental effects of these stabilizing chemicals have been demonstrated.

3. Recommendations

This review of stabilization methods has shown some interesting possibilities for preventing or minimizing erosion. Unfortunately, much of what can be written is based on dry-land technology for resisting rainfall erosion. To confirm the applicability of this technology to the stabilization of dredged materials during re-wetting by tidal action, some pilot work is required.

Small test plots should be developed for testing physical and chemical stabilization techniques as well as combinations of these techniques. These test plots, each perhaps 100 $m^2$, could be developed by placing small amounts of dredged material from the "Pond 3" area in Hayward in a similar area currently exposed to tidal action. With the proper institutional and legal clearance, these plots might be created in the small pond immediately to the north of the northwest corner of Pond 3, or in the part of the Faber Tract site which is presently a mud flat.

### D. Laboratory Study on Movement of Salt through Dredged Material

Since many candidate sites for marsh creation are at old salt evaporation ponds, there is some concern that salt might migrate upward through the deposited dredged material and inhibit plant growth. To evaluate this possibility, an experiment was performed in the laboratory.

The experiment was conducted in a cast acrylic (Plexiglas) column approximately 5 ft. high and 6 in. OD with 1/8 in. wall thickness. A 3 in. salt layer consisting of evaporated sea water* was placed on the bottom and 33 in. of sediment placed on the salt. Sample ports along the column, as shown in Figure 4-5, were used to obtain samples of the sediment for analysis to determine the rate of upward salt migration

*Hain Pure Food Company, Inc., Los Angeles, Calif.

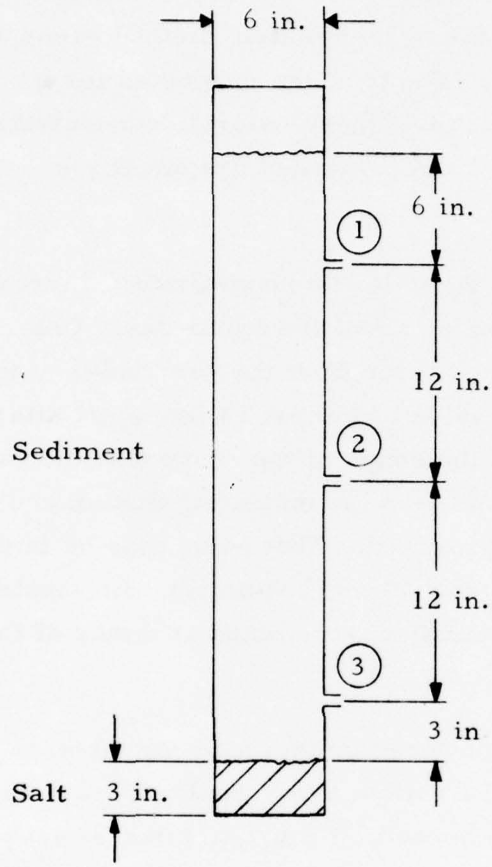


Figure 4-5. Salt Movement Test Column

through the sediment. Sediment samples of approximately 7 ml were mixed in a blender with distilled water to extract the salt filtered to remove the sediment particles, and analyzed for chloride concentration by the argentometric method presented in Standard Methods [18]. The test was conducted for a period of 33 days. The result of these chloride concentration tests is shown in Table 4-2 and presented graphically in Figure 4-6.

After an initial decrease, the chloride concentration increased at the lowest port beginning on about the fourth day. Chloride values at that port increased throughout the remainder of the test period and reached a value of almost 35,000 mg/l after 33 days. It appears from the shape of the curve that the rate of increase was decreasing, perhaps indicating that an equilibrium value was being approached. That value may be in the order of 40,000 mg/l $Cl^-$ (66,000 mg/l salinity). No significant changes in chloride concentration were noted at either of the upper two sample ports.

The questions to be answered by this experiment were first, whether salt from the bottom layer would migrate upward through the sediment, and second, if migration takes place, what the rate would be. During the test period the chloride concentration at the port nearest the salt layer increased to about three times the initial concentration, thus answering the first question. However, determining the rate of migration is more difficult. Expression of the rate could be in terms of the velocity of the front. From Figure 4-6 it appears that between four and seven days were required for the front to reach the lowest port which was 3 in. from the salt layer. If the time were seven days for a noticeable increase to occur,

Table 4-2

Salt Column Chloride Concentration

| | Chloride Concentration, mg $Cl^-$/l | | |
|---|---|---|---|
| Day | Port 1 | Port 2 | Port 3 |
| 0 | 12,300 | 10,300 | 11,200 |
| 4 | 9,000 | 9,000 | 10,100 |
| 7 | 10,000 | 8,800 | 11,500 |
| 14 | 10,100 | 9,500 | 18,200 |
| 21 | 9,200 | 8,800 | 29,900 |
| 33 | 10,100 | 9,200 | 34,400 |

Sample standard derivation = 226 mg $Cl^-$/l at 9200 mg/l

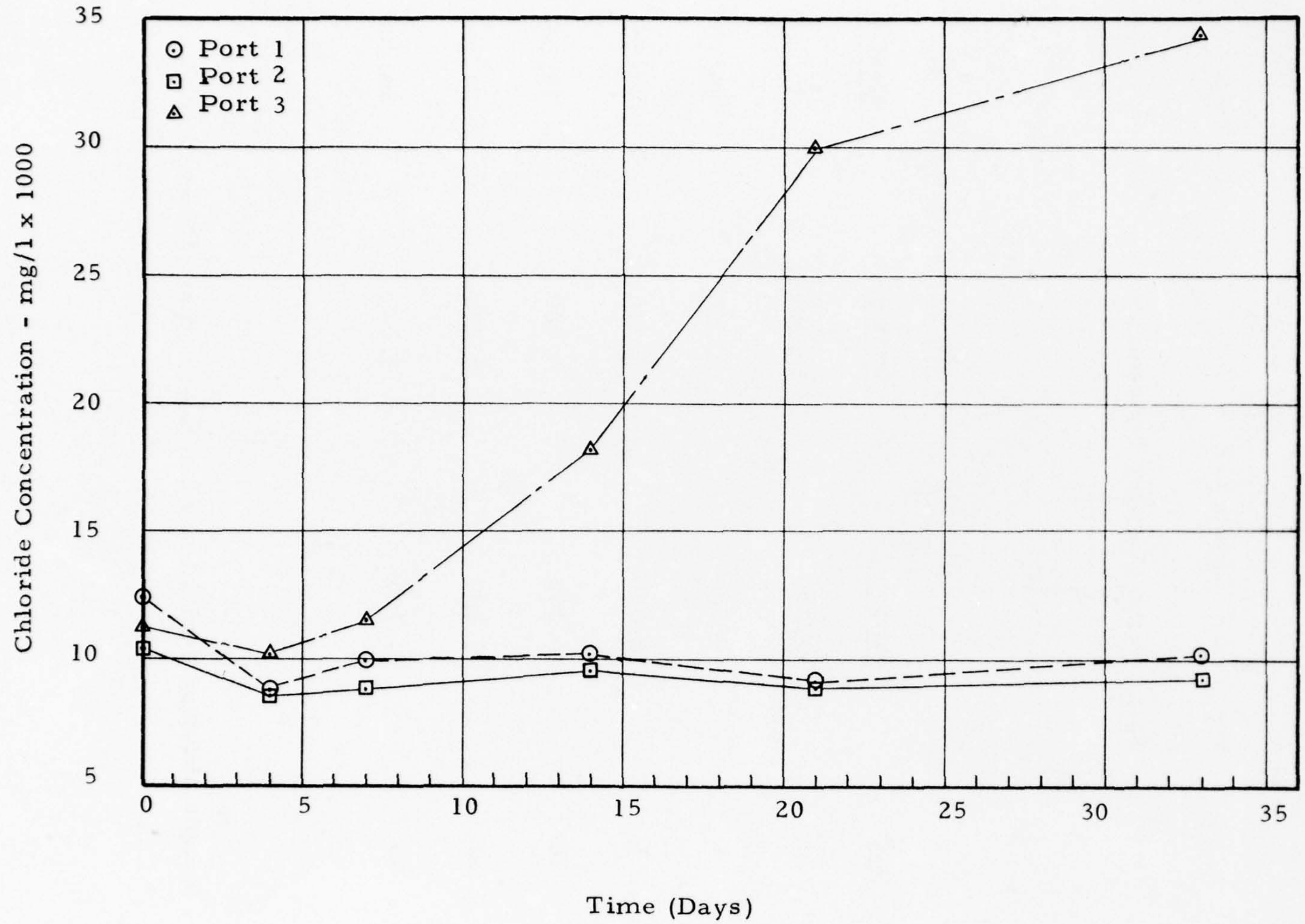


Figure 4-6. Salt Column Chloride Concentration

then the migration velocity was 0.43 in/day. No increase was noted at the second port, but at a velocity of 0.43 in/day it would have required about 35 days for the front to travel that far. Time constraints on the duration of the test required that the test be terminated at 33 days. Thus it is not known with certainty that the front would continue up the column at the rate of about one-half inch per day.

The conditions of this experiment were deliberately restricted and severe, to assure at least a "yes/no" answer on the question of vertical salt migration. A real situation would be less severe, with lower migration rates, for the following reasons:

- Horizontal migration would be possible in a real diked area, and salt could be leached from the old pond bed in this manner.
- Tidal flushing would provide a continual source of dilution water.
- A less thick salt reservoir would presumably exist. Ponds can be expected to be out of use for some time prior to filling, and rain water will cause salt to migrate downward or be carried out through drainage with the rainwater. This was the situation at Pond 3 in Hayward before it was filled for marsh creation.

Nevertheless, the finding that migration did occur in the lysimeter test indicates that a problem may exist. Longer tests should be run to develop more quantitative data. In the absence of such data, and for safety, any salt ponds filled for marsh creation should be allowed to "rinse" for at least one rainy season before filling.

## E. Conclusions

1. Control over fill elevations in marsh creation is important to the establishment and maintenance of desirable marsh plants.

2. Prediction of the elevation of a dredged material fill as it results from future settlement is difficult with the present state of the art. Silts and clays may ultimately settle a distance of more than 20% of the initial fill thickness. Fine-grained materials normally settle more than coarse-grained materials.

3. Conditioning of material with equipment such as bulldozers can accelerate dewatering and consolidation, thus avoiding a long settling period. The fill elevation can thereby be known sooner, resulting in well-informed decisions regarding the possible need for more fill at a site.

4. Tidal inlets develop flow areas such that erosion and accretion are in equilibrium. The mean maximum tidal current velocity of many such inlets is approximately 1 m/sec, and average velocities range from 0.5 to 1 meter per second. These data are for larger inlets than would be needed in a marsh building project, however.

5. Draglines in combination with cable/"deadman" systems are available for excavating tidal inlets up to 1500 feet in length across inaccessible areas.

6. Intertidal stabilization of sediments appears to be most feasible through anchored mats of various natural and synthetic materials, or stabilizing chemicals similar to those used for erosion control on construction projects.

7. Salt does migrate perceptibly through dredged materials from the Bay Area. In the laboratory experiment conducted as part of this study, the upward migration rate of a detectable salt front was approximately 0.4 inches per day, but more detailed simulations would be needed to estimate field conditions.

## F. Recommendations

1. Elevation as a function of time should be monitored on recently-filled confined dredged material disposal areas, and on areas to be filled in the future. Observations should be correlated with soil parameters such as grain size distribution, void ratio, and organic content.

2. When the results of the Waterways Experiment Station, Dredged Material Research Program's Work Unit 4A16 are available ("Development of a Methodology for Determining the Stable Elevation of Confined Dredged Material in Marsh-Creation Efforts," being conducted at Massachusetts Institute of Technology), they should be applied to San Francisco Bay situations.

3. The tidal inlet (dike breach) at Faber Tract in Palo Alto should be studied with respect to its hydraulic properties (channel dimensions, current velocities) to ascertain whether literature data on tidal inlets are directly applicable to the smaller inlets which are needed in marsh-building projects.

4. To learn whether dry-land erosion control measures are suitable for intertidal stabilization, test plots should be established using various plants, mats, and chemical stabilizers on dredged material subject to tidal action.

5. Further salt-migration experiments should be conducted. These experiments should be designed to simulate tidal flushing and to allow horizontal migration. In conjunction with this effort, the soil salinity of any dry evaporation pond beds should be measured before and after the rainy winter season to determine the amount of salt which may be available for migration.

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## V. LAND DISPOSAL OF DREDGED MATERIAL

### A. Introduction

In the past the San Francisco District of the U. S. Army Corps of Engineers has dredged approximately 7 million cubic yards of sediments each year during maintenance dredging activities [1]. Additionally, the Corps also dredges a substantial amount of sediment during new construction. Future dredging has been estimated to average 13 million cubic yards per year for the next 5 years, and between 9 and 10 million cubic yards per year for the following 15 years. The greatest amounts will come from Mare Island (29 percent), Pinole Shoal (17 percent), and Oakland (18 percent) [2]. Some of these sediments contain contaminants resulting from man's activities and thus their disposal may be a difficult and expensive problem.

A method of dredged material disposal which has received considerable attention in recent years is land disposal. However, many questions must be answered before land disposal can be utilized for a significant portion of San Francisco Bay dredged material. A previous study in the research program of the San Francisco District has evaluated the comparative economics of alternative dredging and disposal methods and determined the practicability of land disposal in terms of environmental advisability, technical feasibility, and economics [2]. It is the purpose of the following section of this report to evaluate land disposal operations in terms of the fate of the materials when placed in the disposal area, and treatment processes which may be utilized to minimize the spread of pollutants.

Dredged sediments are made up of materials ranging in size from dissolved ions to objects such as boulders and tree stumps weighing up to several tons. Typically, inorganic minerals will

be present in the form of gravel, sand, silt, and clay.

The presence of some chemicals not found in nature, such as chlorinated hydrocarbons, clearly indicates the input of man's activities. When sediments are polluted so that ocean disposal is unacceptable, a number of alternative disposal techniques are available. Among these are diked disposal possibly including treatment of the dredged material to either concentrate and contain the contaminants, or to destroy or detoxify them. Treatment of dredged materials should not be considered as a technique to be employed with all polluted dredged materials, but rather as another alternative for consideration prior to disposal operations. Establishment of treatment parameters will allow judgements to be made about the optimum method of dredged material disposal considering both environmental and economic factors.

In the following sections dredged material treatment processes which might be used in San Francisco Bay land disposal areas are discussed. As a basis for this review, Bay sediment are characterized and the behavior of pollutants in the environment is described. Treatment processes and systems are then presented which will minimize the environmental impact of dredged material disposal on land.

## B. Dredged Material Characterization

One of the most important considerations in the design and selection of treatment processes is knowledge of the characteristics and behavior of the material to be treated. Firm assessment of treatment process applicability to particular problems is based on treatability data developed in laboratory and field testing. Since the need for dredged material treatment is relatively a new

development, little treatability data is currently available. Considerable information is available enumerating chemical and physical characteristics of San Francisco Bay sediments. Because regulatory efforts to control disposal of polluted sediments are in an evolutionary stage it is difficult in many cases to determine whether a given sediment is polluted. The levels of treatment required are even harder to define.

The first effort at definition of the pollutional nature of dredged material was the 1971 Environmental Protection Agency "Criteria for Determining Acceptability of Dredged Spoil Disposal to the Nation's Waters." These criteria included limits on volatile solids, COD, total Kjeldahl nitrogen, oil and grease, mercury, lead, and zinc. Attempts to work with these criteria produced questions about the technical relevance and adequacy of the procedures. The major difficulties may be summarized as [ 3] :

- "Little or no known correlation exists between the concentration of various chemical constituents within sediments subject to dredging and disposal operations and consequent effects on water quality," and
- "Several of the listed variables, most notably volatile solids and chemical oxygen demand, provide little meaningful information when applied to sediments, especially marine sediments."

Another report [ 3] has stated that the effect of trace metal concentrations on "the environmental impact of dredging is difficult to assess and it must be stressed that bulk metal analyses of sediments is not a useful index of potential environmental water quality problems because the many chemical forms of metals present in the sediment do not have equal impact."

The key then is the availability of pollutants to the environment rather than just the presence. However, review of sediment

quality will provide guidance to the potential for environmental damage in a broad sense and can serve to focus attention on specific problem areas.

An important characteristic of sediments is the particle size distribution. This is particularly important in land disposal operations since these facilities generally include a holding pond in which solids from the dredged material slurry will settle out. Higher percentages of fine material (clay) would indicate both a reduced tendency for sedimentation and, since the total particle surface area is much larger, adsorbed pollutants may also be discharged at a greater rate than for larger particles.

Table 5-1 presents data on the particle size distribution of San Francisco Bay sediments. It is apparent that, although high percentages of sand are present in some of these sediments, clay particles constitute the major portion of most Bay sediments, with silt the next largest portion. The discharge from diked disposal areas may contain a significant portion of these small grained solids and contribute to such problems as a turbidity plume and release of pollutants associated with the solids.

A study by JBF Scientific Corporation for the Environmental Protection Agency [5] reviewed data on sediment quality from throughout the country to locate "hot spots" and set priorities for removal or inactivation of in-place pollutants as required by Title 1, Section 115 of the Federal Water Pollution Control Act of 1972, PL 92-500. Based on consideration of the level of pollutants, area and volume of the hot spots, availability of disposal sites, population served by the waterway, and economic significance of the waterway, it was concluded that San Francisco Bay should be one of the areas of further study and possible rehabilitation. To a large extent this conclusion was based on very high levels of

TABLE 5-1

Particle Size Distribution of San Francisco Bay Sediments [4]

| Location | Sand | Silt | Clay |
|---|---|---|---|
| | Percentages | | |
| Oakland Outer Harbor | 50 | 22 | 28 |
| | 31 | 30 | 39 |
| | 59 | 17 | 24 |
| Oakland Outer Harbor Turning Basin | 2 | 28 | 70 |
| | 6 | 25 | 69 |
| | 3 | 28 | 69 |
| | 11 | 38 | 51 |
| Oakland Inner Harbor | 16 | 26 | 58 |
| | 31 | 21 | 48 |
| | 22 | 33 | 45 |
| Islais Creek Shoal | 6 | 34 | 60 |
| | 7 | 33 | 60 |
| Richmond Harbor Channel | 6 | 34 | 60 |
| | 7 | 33 | 60 |
| Southampton Shoal Channel | 91 | 5 | 4 |
| | 91 | 3 | 7 |
| Pinole Shoal Channel | 12 | 38 | 49 |
| | 9 | 33 | 58 |
| Mare Island Strait Channel | 6 | 43 | 51 |
| | 8 | 38 | 53 |
| | 8 | 41 | 51 |
| | 12 | 46 | 42 |
| Redwood Creek Channel at the Mouth | 8 | 35 | 57 |
| | 5 | 37 | 58 |
| | 6 | 35 | 59 |
| Redwood Creek by Corkscrew Slough | 12 | 31 | 56 |

cadmium, lead, and copper reported in the Islais Creek Channel. Since the data indicating high values is very limited, additional sampling was recommended.

A more complete overview of San Francisco Bay sediment quality is provided by other data gathered for that JBF report and summarized in Table 5-2. For gross purposes of characterizing these sediments, the analyses may be compared to old EPA sediment guidelines [1]. These guidelines are not presently used by regulatory agencies, having been replaced by elutriate testing procedures. They do, however, provide a rough measure against which the bulk analyses of Table 5-2 may be compared. When the sediment analyses are compared to the EPA dredged material disposal criteria presented at the end of Table 5-2, it is seen that the limit on volatile solids is generally exceeded in each of the areas presented. COD and TKN values are often exceeded and oil and grease values are relatively low. Lead values are, on the average, approximately at the level of the EPA standards, mercury values are generally less, and zinc values considerably greater. These levels are typical of industrialized harbors and do not represent gross pollution. Individual, limited locations may exist within the Bay where point sources or natural deposits have produced higher concentrations of contaminants, but in general the pollutant content of San Francisco Bay sediments is not excessive.

## C. Treatment Objectives

The objective of land disposal of dredged material is to provide sufficient storage capacity at a reasonable cost without causing degradation of water quality or other environmental problems. Many water treatment and solids handling and treatment processes are available which may assist in accomplishing this land disposal objective. In general, these treatment processes may be considered as:

## TABLE 5-2

### Pollutant Concentrations in San Francisco Bay Sediments

(Data provided by San Francisco District of Corps of Engineers, by San Francisco Port Commission, and by Richmond Department of Public Works)

| Location | Pollutant | Concentration Value | Concentration Unit | Number of Analyses |
|---|---|---|---|---|
| San Francisco Dock Area | Vol. Solids | 7.1 | percent | 23 |
| | COD | 4.0 | percent | 23 |
| | TKN | 0.17 | percent | 23 |
| | Oil & Grease | 0.05 | percent | 23 |
| | Cd | 0.38 | mg/kg | 16 |
| | Cu | 52 | mg/kg | 23 |
| | Pb | 12 | mg/kg | 23 |
| | Hg | 0.82 | mg/kg | 16 |
| | Zn | 136 | mg/kg | 23 |
| Mare Island Strait | Vol. Solids | 8.0 | percent | 36 |
| | COD | 5.2 | percent | 36 |
| | TKN | 0.07 | percent | 36 |
| | Oil & Grease | 0.07 | percent | 36 |
| | Cd | 3.3 | mg/kg | 33 |
| | Cu | 87 | mg/kg | 47 |
| | Pb | 60 | mg/kg | 47 |
| | Hg | 0.3 | mg/kg | 47 |
| | Zn | 218 | mg/kg | 47 |
| Alameda Naval Air Station | Vol. Solids | 7.7 | percent | 28 |
| | COD | 6.5 | percent | 23 |
| | TKN | 0.19 | percent | 28 |
| | Oil & Grease | 0.10 | percent | 21 |
| | Cd | 1.0 | mg/kg | 15 |
| | Cu | 53 | mg/kg | 20 |
| | Pd | 54 | mg/kg | 28 |
| | Hg | 0.56 | mg/kg | 23 |
| | Zn | 146 | mg/kg | 28 |
| Richmond | Vol. Solids | 7.0 | percent | 22 |
| | COD | 4.8 | percent | 22 |
| | TKN | 0.11 | percent | 22 |
| | Oil & Grease | 0.14 | percent | 22 |
| | Cd | 3.3 | mg/kg | 20 |
| | Cu | 65 | mg/kg | 22 |
| | Pb | 46 | mg/kg | 25 |
| | Hg | 1.1 | mg/kg | 25 |
| | Zn | 139 | mg/kg | 15 |

TABLE 5-2 (continued)

Pollutant Concentrations in San Francisco Bay Sediments

| Location | Pollutant | Concentration Value | Concentration Unit | Number of Analyses |
|---|---|---|---|---|
| Islais Creek | Cd | 0.9 | mg/kg | 5 |
| | Cu | 19 | mg/kg | 5 |
| | Pb | 13 | mg/kg | 17 |
| | Hg | 0.6 | mg/kg | 17 |
| | Zn | 42 | mg/kg | 5 |
| Oakland Harbor | Cd | 1.1 | mg/kg | 6 |
| | Cu | 53 | mg/kg | 6 |
| | Pb | 53 | mg/kg | 16 |
| | Hg | 0.5 | mg/kg | 16 |
| | Zn | 152 | mg/kg | 16 |

Criteria Formerly Used by the
Environmental Protection Agency
for Disposal of Dredged Material [1]

| Pollutant | Concentration Value | Concentration Units |
|---|---|---|
| Vol. Solids | 6.0 | percent |
| COD | 5.0 | percent |
| TKN | 0.10 | percent |
| Oil & Grease | 0.15 | percent |
| Pb | 50 | mg/kg |
| Zn | 50 | mg/kg |
| Hg | 1.0 | mg/kg |

- methods for effluent water quality control and minimizing the release of contaminants, and
- methods for extending the useful life of the dredged material disposal site.

1. Effluent Water Quality Control

The San Francisco Bay Region of the California Regional Water Quality Control Board is responsible for setting water quality regulations for dredged material containment areas on San Francisco Bay. A recent representative set of discharge specifications included the following limitations [6]:

a. Turbidity at a point 100 feet from the discharge point may not be increased by more than 5 units if the background is less than 50 units, by more than 10 units if the background is 50 to 100 units, and more than 10 percent of background if the background is greater than 100 units.

b. The amount of settleable matter may not exceed 1.0 ml/l/hr.

c. The concentration of lead may not exceed 0.05 mg/l, and mercury may not exceed 0.005 mg/l.

To meet these requirements it will be necessary that sound engineering procedure be followed in the design of the containment area. Limitations on turbidity and settleable solids can generally be met by providing adequate residence time in a holding pond. Settling tests have been performed for the Buffalo District of the Corps to provide an estimate of the amount of pollutants which would be discharged from diked disposal areas [7, 8]. Samples of hopper dredgings were allowed to settle for periods of one to 40 hours followed by measurement of physical and chemical parameters of the supernatant liquid. A typical result is shown in Table 5-3. In eight samples tested, settling for even one hour produced dramatic results. The removal of suspended solids was in each case greater than 99 percent, leaving residual concentrations

## TABLE 5-3

### Settling Test Data, Fairport Harbor, Ohio [7]

| | INITIAL | ONE HOUR SETTLING | FOUR HOUR SETTLING | EIGHTEEN HOUR SETTLING | FORTY HOUR SETTLING |
|---|---|---|---|---|---|
| COD, mg/l | 14,540 | 202 | 133 | 97 | 101 |
| TOTAL SOLIDS, mg/l | 363,000 | 4240 | — | 2712 | 2460 |
| DISSOLVED SOLIDS, mg/l | 2140 | 2140 | 2140 | 2140 | 2140 |
| SUSPENDED SOLIDS, mg/l | 360,860 | 2130 | — | 672 | 320 |
| TURBIDITY, JTU | 155,000 | 780 | — | 195 | 52 |
| TOTAL PHOSPHORUS, mg/l | 10 | 0.280 | — | 0.58 | 0.16 |
| SURFACTANTS, mg/l | — | 0.06 | — | — | — |
| SEDIMENT, ml/l | — | 700 | — | — | — |
| KJELDAHL NITROGEN, mg/l | 696 | 56.5 | — | 33.2 | — |
| OIL AND GREASE, mg/l | 1865 | 4 | — | <10 | <10 |
| IRON, mg/l | 6500 | 22.5 | — | — | — |
| CHROMIUM, mg/l | 73 | 0.200 | — | 0.040 | 0.035 |
| CADMIUM, mg/l | <0.4 | 0.030 | 0.052 | 0.051 | 0.048 |
| ZINC, mg/l | 29 | 0.150 | 0.099 | 0.025 | 0.025 |
| MANGANESE, mg/l | 145 | 2.200 | — | 0.021 | 0.019 |
| COPPER, mg/l | 11 | 0.070 | — | 0.060 | 0.070 |
| NICKEL, mg/l | <3.6 | — | — | <0.015 | — |
| CHLORIDES, mg/l | — | 284 | — | — | — |

between 210 and 2130 mg/l. Although further treatment may be necessary prior to discharge of the supernatant, the sedimentation process will produce an effluent having a low suspended solids concentration and low in other pollutants.

A second aspect of return flow quality is the concentration of metals. That concentration is very difficult to predict because the complex nature of the chemical reactions and physical interactions between the metals, water, particulate matter, organics, and other influences make estimates of water quality uncertain. The following factors will all interact to determine the metals concentration in return flows:

a. Particulate Matter. The association of heavy metals with particulate and colloidal species can be attributed to a variety of mechanisms. Sand and silt particles contain minerals which may include heavy metals, but should not contain large concentrations of adsorbed metals due to their limited surface area. Clay minerals will contribute heavy metals both by incorporation within the mineral structure and by adsorption and ion exchange onto the surface and into the structure.

The adsorption of trace elements onto mineral surfaces can occur without ion exchange as a result of coulombic interactions with the surface. Ion exchange can also exert a considerable effect on trace metal uptake by suspended particulates and is considered to be the most important mechanism for chemical control at the sediment-water interface in the oceans [ 9] .

b. Solubility. In some cases solubility should be the limiting factor on heavy metal concentrations, but there is considerable uncertainty in the chemical nature and composition of the precipitate [ 10] . A number of elements are often found in concentrations which exceed solubility limits. However, the

limits are based on equilibrium considerations and natural waters are often not at equilibrium. An additional factor is the possibility of coprecipitation which would result in entrapment of some metals by the precipitation of others.

c. Complexation. A potentially important factor affecting heavy metal concentrations is complexation by both organic and inorganic ligands [10]. Properties of the heavy metals which may be influenced by formation of complexes include solubility, adsorption on suspended and bottom sediments, and uptake by biota.

d. Oxidation-Reduction. Microbial degradation of organic matter causes lowered oxygen levels, lowered redox potentials, and lowered pH which increases the solubility of many metals [11]. Iron is known to be important in this way. In a diked disposal area metal concentrations have been found to first decrease and then increase with time [12]. The mechanism proposed was that reduced iron in the dredged material was released and oxidized in the water during dredging. The resulting insoluble ferric hydroxide precipitated, with the precipitate carrying other metals to the bottom by adsorption and/or entrapment. When the iron was re-reduced in the sediments forming soluble ferrous iron, the heavy metals were released back into the water.

Pratt and O'Connor have considered the probable effect on metal mobility of oxygen levels in sediments [11]. In general, they concluded that in oxygenated sediments the formation of insoluble compounds and sorbtion results in a low flux to the overlying water. If the sediments are anoxic, that chemical environment alters metal species to produce increased mobility. On the other hand, if hydrogen sulfide is produced by anaerobic microorganisms, then the sulfide will precipitate many metals as the very insoluble

sulfide. Copper, zinc, and cadmium should be immobilized under anoxic conditions, but cobalt, manganese, and nickel are readily soluble. The behavior of mercury is more complicated because under anaerobic conditions mercury compounds can be methylated to the volatile, highly toxic methylmercury. However, if hydrogen sulfide is present, then insoluble mercury sulfide may be the result.

A recent study conducted for the San Francisco District of the Corps attempted to determine the extent to which sediment-associated pollutants may be released to the water column and made available to the ecosystem as a result of maintenance dredging and disposal activities in the San Francisco Bay and Estuary [4]. Conditions investigated included such sediment characteristics as organic content, cation exchange capacity, and minerology and also salinity, pH, Eh, temperature, and pollutant concentration in both the water and sediments. It was concluded that, for San Francisco Bay, the oxidation-reduction potential appeared to be the most significant environmental parameter which controlled the final water column metals concentration after mixing water with polluted sediment. When the mixing was accomplished under oxidizing conditions the concentration of cadmium, copper, lead, and zinc all showed significantly higher concentrations. Under reducing conditions and in the presence of sulfides these metals would be very susceptible to incorporation into the sediment as insoluble sulfides. Apparently when under oxidizing conditions the presence of the hydrous oxides of iron, aluminum, and manganese were relatively ineffective in preventing release of heavy metals.

Iron and manganese operated in the opposite manner in that they were released under reduced conditions and held under oxidizing conditions. This is due to the increased solubility of the lower

valence species compared to the oxidized species. It is evident that the concentration of dissolved oxygen in the containment area water and sediment will have a pronounced effect on the mobility of metals.

Dredged material containment areas waters generally contain relatively high concentrations of dissolved oxygen as a result of photosynthetic production by algae and dissolution from the atmosphere. It is probably true that in most cases there is insufficient oxidizable organic matter in dredged material to produce anoxic conditions by bacterial action in the water of diked disposal areas, although the sediments will exist in an anoxic state except for an aerobic zone which may occur in the top few centimeters.

An attempt has recently been made [4] to develop predictor relationships to estimate the concentrations of heavy metals that would result on mixing of sediment with water. The predictions which resulted generally produced imprecise estimates which were not of appreciable value in determining soluble metal concentrations. In diked disposal areas additional complications enter because the water storage time is variable and uncertain due to gradual filling of the site and hydraulic short circuiting. Also, in addition to soluble species, metals would be associated with any particulate matter escaping in the effluent.

In summary, the possible release of metals from diked disposal areas depends both on the chemical nature of each metal and also on the presence of oxygen and sulfide in both the sediment and overlying water. It is very difficult to predict the level of metals discharged from a diked disposal area. It is tempting to try to estimate metal concentrations in the return flow, based on bulk sediment analyses, but it has generally been concluded that bulk

chemical composition is not a useful index of potential environmental water quality problems. This is because bulk analyses do not account for the varying impact of the different chemical forms of an element [4], [12].

Another approach to estimating the concentration of metals in diked area return flows would be through evaluation of the return flows from existing diked disposal areas. Unfortunately, land disposal is not widely used in the San Francisco Bay area at this time so that existing records of return flow quality must be obtained from other geographical areas. Those sediments are different in chemical and physical composition and the disposal areas may not have been designed by good engineering standards. Nevertheless, an indication of return flow quality can be obtained.

Windom [13, 14] has presented the data in Table 5-4 on confined disposal areas in southeastern United States. Where multiple sets of data are given, samples were analyzed at several times at the same discharge point. Metal values are seen to be somewhat variable, but quite low in comparison to the San Francisco Bay standards previously presented (Pb 50 $\mu g/l$, Hg 5$\mu g/l$). This research indicates that properly designed settling areas are capable of producing effluents which can meet the existing water quality standards of the California Regional Water Quality Control Board.

TABLE 5-4 - Metal Concentrations in Diked Area Return Flows [ 13, 15 ]

| | Metals (all values in μg /1 | | | | | |
|---|---|---|---|---|---|---|
| | Fe | Cd | Cu | Pb | Hg | Zn |
| Brunswick Harbor (Georgia) | 80 | < 0.1 | 22 | 0.5 | 0.3 | 0.5 |
| | 70 | < 0.1 | 14 | 0.9 | 0.4 | 1.0 |
| Cooper River (South Carolina) | 60 | 3.0 | 10.5 | 3.4 | --- | --- |
| | 22 | 0.6 | 3.8 | 2.3 | --- | --- |
| | 60 | 0.4 | 6.0 | 2.6 | --- | --- |
| Terry Creek (Georgia) | 47 | 1.0 | 5.0 | 9.5 | --- | --- |
| Savannah River (Georgia) | 28 | 0.21 | 24 | 8.8 | --- | 29 |
| | 31 | 0.05 | 32 | 8.4 | --- | 23 |
| | 19 | 0.05 | 31 | 8.4 | --- | 23 |
| | < 100 | ---- | 8 | --- | 0.27 | 19 |
| | < 100 | ---- | 6 | ---- | 0.16 | 9 |

If sedimentation alone will not result in a satisfactory effluent due to poor settling properties of the solids or insufficient detention time, coagulant chemicals may be used. Coagulation is a process employed for the separation from water of materials in colloidal form or in a fine suspension. A practical definition of colloids is those particles which will not settle in a reasonable amount of time. The cause of their stability is that they are too small to settle rapidly and will not agglomerate to form large particles due to similar electrostatic surface charges.

Measure of coagulation efficiency is often expressed in terms of the percent removal of suspended solids or turbidity. Many other classifications of pollutants are also associated with suspended matter, so that as turbidity is reduced, concentrations of coliform bacteria, chemical oxygen demand biological oxygen demand, particulate heavy metals and other pollutants are also reduced.

An important difference in the coagulation mechanism exists between the two types of coagulants, inorganic metal salts and synthetic polymers. Polymers act by forming "bridges" among particles to tie them together. The effect is completely physical, and therefore dissolved pollutants are not affected. On the other hand, metal salts (alum, lime, and iron compounds) enter into chemical precipitation reactions with other pollutants such as phosphorus and dissolved metals.

Coagulation systems consist of three operations: 1) mixing of coagulant chemicals with the wastewater, 2) flocculation, or slow mixing, which causes collisions among the destabilized particles, and 3) sedimentation to remove the large floc particles which are formed. Filtration may also be used either in place of, or in addition to, sedimenation.

Many chemicals may be employed in the coagulation process. Each of these behaves differently depending on the concentration used, the pH during coagulation, the nature of the colloids, and the nature of the water. Determination of the type and quantity of chemical and the best pH value is based on a series of laboratory batch tests called "jar tests". Since little is known about coagulation of dredged materials, extensive testing would be required prior to coagulation design.

A study performed for the Buffalo District by Dow Chemical Company evaluated the use of synthetic polymers to treat the effluent from a diked disposal area [ 15 ] . Laboratory coagulation tests were performed with return water from two disposal sites, one of which contained a high suspended solids concentration, since the capacity of the disposal area had almost been exhausted. The other had a very low suspended solids concentration. It was concluded that the best coagulation was accomplished with ferric chloride and a Dow polymer, Purifloc C-31. This combination was then tried in field tests at that dredged material disposal site which had the low suspended solids concentration to demonstrate that coagulation could further reduce the pollutant load in the disposal area discharge. The chemical dose was estimated to be 33 mg/l $FeCl_3$ and 7.5 mg/l Purifloc C-31. Flocculation was accomplished in a sloped corrugated metal pipe. The settling area was a diked pond having a theoretical detention time of about 10 hours. It was found that suspended solids concentration could be reduced from 55 mg/l to 26 mg/l (52 percent removal) and that turbidity could be reduced from 97 units to 26 units (73 percent removal). A reduction in total coliform was also noted, but total phosphorus and COD remained essentially the same. Since these measurements were taken at the point of discharge rather than 100 feet from that point and the background turbidity was not stated, it is not possible to directly relate this degree of removal

to San Francisco Bay water quality requirements. However, it is clear that coagulation systems can substantially upgrade land disposal area effluent quality in a simple and inexpensive system.

A second method for removal of suspended solids in disposal area effluent is fine screens. Fine screens may be of the disk or drum type, with stainless steel or non-ferrous wire-mesh screencloth. The disk type has a vertical circular screening surface that rotates on a horizontal shaft set slightly above the water surface. The drum type revolves at about 4 rpm around a horizontal axis and operates about half-submerged. The water flows in one end of the drum and outward through the screen cloth. Commonly used screen openings are $23\mu$, $35\mu$, and $60\mu$. In both the disk and drum types the solids are raised above the liquid level by rotation of the screen and are back flushed into receiving troughs by high pressure jets. One style of drum type fine screen is called a Microstrainer which is a trademark of Crane Company, but other manufacturers offer similar equipment.

Similar to fine screens are filtration devices which also are used to remove suspended solids from water. Numerous filtering systems are available from equipment manufacturers. One interesting concept is the moving bed filter by Johns-Manville Products Corporation. The sand filter medium is driven through a cone counter-current to the flow of wastewater. Movement of the sand is accomplished by a hydraulically actuated diaphragm. Since the sand is constantly being removed, cleaned, and returned to the system, the filter unit does not have to be stopped for backwashing as do conventional units. Another type of unit which is offered by several companies is designed to minimize the backwashing problem by automatically running through a backwash cycle when the head loss reaches a predetermined value.

In addition to automatic gravity filters, numerous pressure filters are available. These consist of a closed steel shell containing a bed of granular filter media over a collector system.

Of these processes for removing suspended matter from water, the process which will be most likely to be employed is plain settling in ponds. If insufficient due to poorly settling solids, inadequate pond volume, or very stringent effluent requirements, then a coagulation system would almost certainly produce an effluent of high enough quality and at a lower cost than either screens or filters.

In urban areas many dredged materials will contain high concentrations of pathogenic organisms due to local sewer and stormwater outfalls. Unfortunately, very little information exists concerning the bacterial pollution potential of dredging contaminated sediments. An indication of the problem is given by data from the Norfolk, Virginia, Craney Island diked disposal site. Limited data on the sampling of the effluent showed that the concentration of total coliform varied between 100 and 33,000 coliform/100 ml. For comparison to water quality uses, coliform limits at bathing beaches usually range from 240 to 2400 coliform/100 ml, and approved shellfish areas generally allow less than 70 coliform/100 ml for a median value with not more than 10 percent exceeding 230 coliform/100 ml [16].

The conditions at Craney Island are almost certainly less severe than those found at other disposal sites since Craney Island has long residence times due to the very large size of the pond area. The efficiency of these settling areas is good since the effluent suspended solids levels during the tests cited above were only 14 to 174 mg/l. In other instances where less pond residence time is available, greater coliform densities should be expected.

Chlorination is by far the most common method of disinfecting water and should be employed if bacterial problems occur in the return flow. A problem with chlorine is that it will rapidly react with any ammonia present. Data from Skidaway Institute of Oceanography [13] has indicated that large quantities of ammonia are released on initial dispersion of polluted sediments. On storage in a containment pond the microscopic plant community in the water will utilize the ammonia and convert it to nitrates. Ammonia concentrations as high as 6 to 8 mg/l were found in diked areas in the Charleston, South Carolina, area. Since it has been found that two molecules of chlorine are required to completely react with each molecule of ammonia, this would represent a "chlorine demand" of 12 to 16 mg/l and would increase the chemical cost of disinfection proportionately.

During the conduct of this study sediment samples were taken at Mare Island to evaluate the potential leaching of pollution from a land disposal site in the San Francisco Bay area. Sediments are disposed of at the Mare Island land disposal area exclusively by the Mare Island Naval Shipyard which operates their own dredge in Mare Island Strait. Sediment is pumped by the dredge directly to the disposal site which is presently filled to a depth of 6 to 8 feet.

A number of sediment samples were taken in the vicinity of the effluent weir near the northerly end of site both inside the site and in the adjacent marsh area as shown in Figure 5-1. It was apparent from visual observations of the topography of the marsh area and cores taken, and confirmed by the Naval personnel who operate the area, that during construction much dredged material washed out onto the marsh and covers the virgin material.

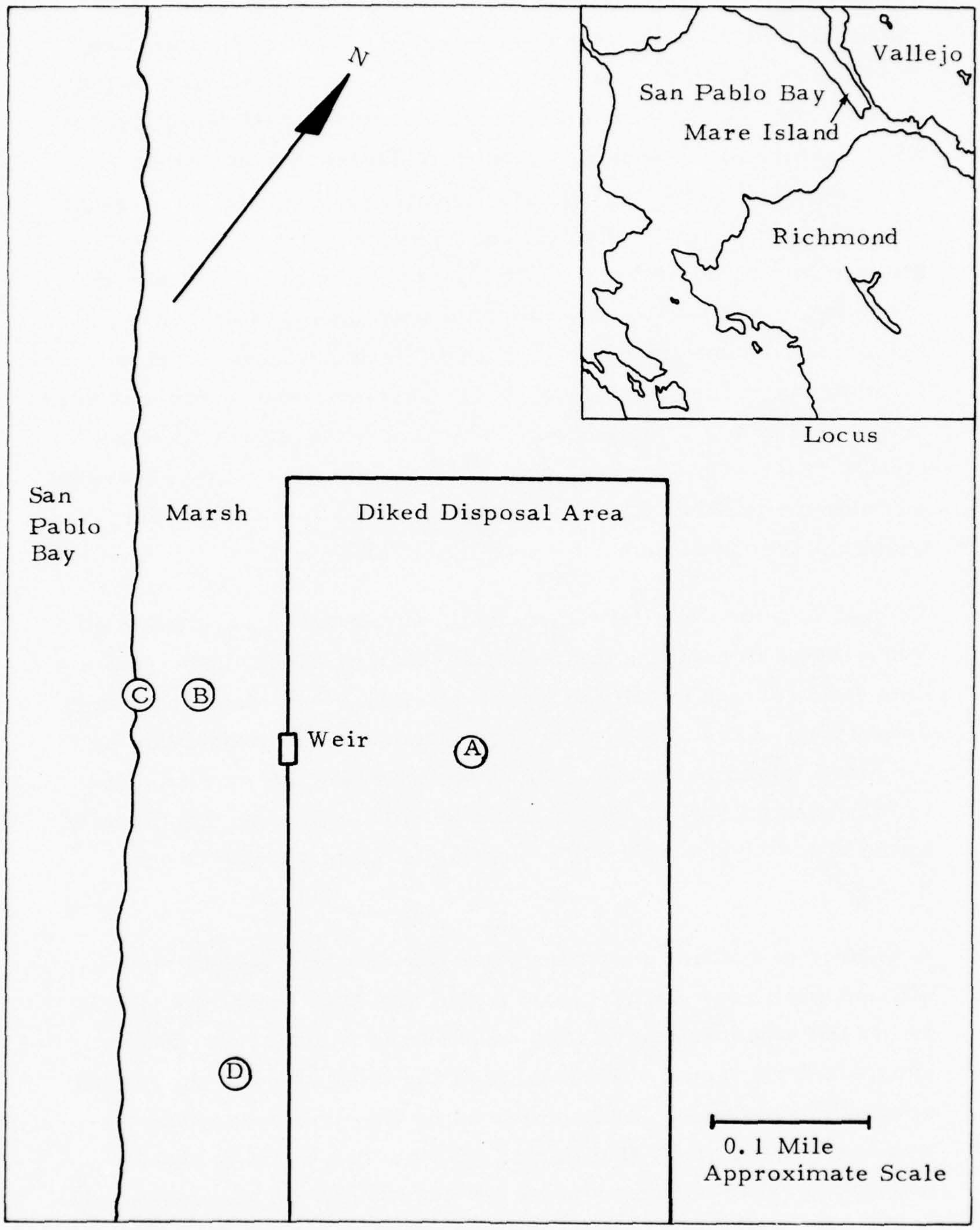


Figure 5-1. Sample Locations at Mare Island Land Disposal Site

A Davis peat corer was used to obtain samples from depths up to 15 feet. The results of analysis of these samples is shown in Table 5-5. Sample point A was in the center of the area approximately equidistant from the east and west dikes. Point B was halfway between the dike and the waters edge where point C was located. Sample location D was located approximately 0.2 miles south of the weir. At point A samples were taken at four depths from the surface to 15 feet to characterize the nature of the material within the disposal site. These dredged materials are seen to be well compacted and quite uniform. Metals levels appear to be slightly higher toward the surface, but the differences are small. Material below original bottom of the disposal area, below about 8 feet, contains essentially the same concentration of metals as that above it. It is not known whether this is due to leaching of metals from dredged material into the virgin material, or on the other hand, a result of the similarity between the virgin material and Mare Island Strait sediment. The disposal site is on a salt marsh and thus probably geologically quite similar to the sediments deposited in Mare Island Strait. A difference is that the recently deposited sediments have been subjected to pollution. Probably both factors have contributed to the similarity in metal content.

When these metal concentrations are compared to the data previously presented in Table 5-2 for Mare Island Strait sediments it is seen that the concentrations of zinc and copper are quite similar, but that cadmium and lead are considerably lower in the disposal area and mercury is at least somewhat lower. Under anoxic conditions, as seen by the concentration of total sulfide which increases with depth, these metals should be precipitated as the sulfide and would not have leached from the sediment. The reason for these unusually low values is not known.

TABLE 5-5 - Characteristics of Mare Island Sediments

| Location* | Sample Depth | Solids % | Total Sulfide mg/kg | Fe % | Mn mg/kg | Metals Zn mg/kg | Cu mg/kg | Cd mg/kg | Hg mg/kg | Pb mg/kg |
|---|---|---|---|---|---|---|---|---|---|---|
| A | Surface | --- | --- | 5.20 | 915 | 232 | 109 | < 0.25 | < 0.16 | < 2.5 |
| | 3 ft | 48.0 | < 1.0 | 4.94 | 1011 | 226 | 107 | < 0.24 | < 0.19 | < 2.4 |
| | 8 ft | 41.6 | 21.1 | 3.70 | 775 | 173 | 79 | < 0.29 | < 0.19 | < 2.9 |
| | 15 ft | 52.2 | 39.4 | 3.84 | 589 | 185 | 87 > | < 0.34 | < 0.19 | < 3.4 |
| B | surface | --- | --- | 4.33 | 713 | 206 | 100 | < 0.25 | < 0.19 | < 2.5 |
| C | surface | --- | --- | 4.12 | 1031 | 206 | 88 | < 0.25 | < 0.2 | < 2.5 |
| D | 1.5 ft | --- | --- | 4.24 | 638 | 226 | 99 | < 0.24 | < 0.24 | < 2.4 |
| | 3 ft | 42.8 | < 1.0 | 4.60 | 1108 | 212 | 96 | < 0.24 | < 0.19 | < 2.4 |
| | 6 ft | --- | --- | 5.20 | 1922 | 232 | 106 | < 0.25 | < 0.19 | < 2.5 |

*See Figure 5-1 for Sample Locations.

Samples B, C, and D were taken outside the disposal area dike. Each shows similar results to those found inside the disposal area. At location D the top two samples are very similar in metals content to each other and to all other samples. At 6 feet, in what must be virgin material, the concentration of iron was slightly higher and manganese was almost twice as high as any other value found. It is not apparent why the level should be so high, but it does not seem to be due to proximity to the disposal area. If it were, then a high manganese concentration would also have been apparent at 3 feet and 1.5 feet.

A sample obtained, but not analyzed, from north of the disposal area again appeared to consist of several feet of dredged material overlying virgin sediments. It is apparent that construction of the disposal area and subsequent filling have resulted in an overlay of dredged material over an area many times greater than the actual disposal site.

Based on this data it does not appear that metals are leaching from the disposal area. The virgin material is very similar in metals content to the dredged material and variations which exist do not appear attributable to the land disposal operation.

2. <u>Methods for the Extension of the Useful Life of Land Disposal Sites</u>

At the present time almost all land disposal sites are operated as dumping grounds at which little attention has been given to optimizing the use of the site so that life of the site is extended for as long as possible. Such methods might include separation of reusable materials and optimization of drying the solids to increase storage capacity. Separation of various types of solids might include segregating sand and gravel for sale to the construction industry, clay for manufacturing of bricks or

aggregate, or treatment of the solids to reduce the level of pollutants. Land disposal area storage capacity can be increased by improvements in solids drying and by other techniques.

As dredged material arrives at a land disposal site, the first type of operation which would be employed would be a separation of large solids from finer particles. The separation may be based on the dimensions of the particles as in screening, or on a specific gravity difference between the particle and water, as in classification.

a. Bar Screens

Bar screens are intended to prevent large objects from entering the treatment process where they would either interfere with later processes or damage equipment. In many cases, hydraulic dredges are able to pick up and transport such items as boulders, scrap metal, wood, etc. which should be removed before any other treatment processes.

b. Sand and Gravel Separation

In dredging projects sand and gravel may make up a large portion of the sediments. Separation of these materials would lessen the volume requiring subsequent treatment, decrease wear and abrasion of equipment, and may provide an opportunity for sale of construction materials,

The characterization data presented earlier in this chapter indicates that although most Bay sediments contain only a small fraction of sand, sediments from some locations are predominately sand (Southampton Shoal Channel, Oakland Outer Harbor). If it can be determined that there is sufficient sand of the proper grain sizes, and that markets exist, then, in addition to the possibility that revenue may be generated, the effective life of the disposal site will be extended in proportion to the amount of material sold.

Processes which have potential for sand and gravel separation are coarse screens, classifiers, and hydrocyclones.

(1) Coarse Screens

Coarse screens are those with relatively large openings which are intended to remove only floatables or large grained, probably inorganic solids. These screens are usually either stationary or move in a circular vibrational or reciprocating horizontal motion.

Three types of coarse screens can be identified. The first of these is vibrational screens which operate by inducing vibrations by rotation of eccentric weights on the end of a motor shaft. These devices may be equipped with several decks of screens to accomplish sizing and classifying into groups of comparable sizes. Units which are designed to remove oversize particles in a high throughput process are called scalpers, and are often used in sand and gravel operations. Such screens may accomplish separations of extremely fine particles down to 400 mesh.

The second type of coarse screen is the stationary screen. A number of non-moving screens, or simple sieves, are on the market which are designed to accommodate high liquid flow rates. The majority of these consist of horizontal bar screens which are inclined at about $45^{o}$ from the vertical. Flow enters from a head box and falls downward onto the screen. Water and fine solids pass through with particles larger than the mesh size collecting at the base of the screen.

The third type of coarse screen is a horizontal rotary drum screen. Water enters from a head box and passes through the screen. Solids are conveyed across the top of equipment by the rotation of the drum and deposited at the far side.

Considerations in the use of coarse screening devices include the positive aspects of simple operation and low space requirements, and the negative aspects of high head required for operation (5 to 6 feet) and the possible noise problem of vibrating screens.

(2) Classifiers

Classifiers are designed to separate large quantities of relatively large and dense solids from water. The operating principle is differential sedimentation in which control of water velocity and residence time allows separations of particles based on settling velocity. This in turn is related to particle size and specific gravity. The principle differences among classifiers are in the methods for collection and transport of the settled solids. Among the types of classifiers are grit chambers and other short residence time settling basins used by industry. Grit chambers have been used for many years in sewage treatment plants as a protective device to remove sand and other abrasive materials.

A device designed for the removal of large quantities of dense solids from water is the wet classifier and consists of a sedimentation tank equipped with a large spiral rake which pushes the settled solids up an inclined plane. Consideration of spiral diameter, pitch, length and speed, and of tank style and pool depth produces the desired mesh of separation.

(3) Hydrocyclones

The hydroclone is an inertial separator in which solids heavier than water are separated by centrifugal forces. Water containing suspended solids enters tangentially at the velocity of 50 to 100 feet per second. Centrifugal forces hold the solids near the outer wall as the fluid spirals downward into a conical section. As the swirling flow enters the converging conical section a secondary flow builds up which carries water inward and upward along the axis to the overflow outlet. Due to the outward centrifugal forces

particles more dense than water tend to move in the opposite direction and exit by the under flow outlet. The separation characteristics of the hydrocyclone are dependent on size, shape, and density of the solids and the hydrocyclone geometry. An investigation conducted at Oklahoma State University (17) to determine whether hydrocyclones may be successfully used to treat dredged materials concluded that clarification and concentration performance of hydroclones was below average to poor, but that the application of hydroclonic devices to classify materials for the recovery of sand was a distinct possibility.

Following separation of the larger grained particles will be a process to separate water from fine grained solids. These processes include gravity settling, coagulation, filtering, and screening, each of which has been considered previously.

A large portion of the water in the dredged material slurry can be readily separated from the solids by a process such as gravity settling. The water which remains is more difficult to separate and requires other processes for its removal. Air drying has often been used for this purpose. Drying actually occurs by two mechanisms, filtration and drying. Most of the water leaves by drainage making the provision of an underdrain system desirable for rapid dewatering. During this filtration period which will last only a few days, the solids concentration may increase from a few percent to 13 to 22 percent. Drying by evaporation occurs at first at a constant rate similar to the rate of evaporation from a free water surface. After a critical moisture concentration is reached, moisture loss occurs at a declining rate during which the final equilibrium solids content, perhaps 25 to 50 percent, is approached exponentially. The drying and removal cycle may take place in 30 to 90 days. In addition to solids and media characteristics, temperature, air

movement, relative humidity, and precipitation will affect the length of time required to attain a given solids content.

Several recent studies have investigated methods for improving utilization of diked disposal areas for the containment of dredged materials. The engineering properties of polluted dredged material have been investigated by Krizek, et al (18) to evaluate their usefulness as landfill materials. They concluded that, when properly dewatered, these materials are similar to fine-grained organic soils and can be compacted to form a medium-density landfill which may be used for parking lots, residential construction, or recreational parks. Drainage of the dredged materials was investigated under several conditions: gravity only, gravity plus vacuum with air pressure being applied directly to the surface of the dredgings, and gravity plus vacuum with a membrane between the surface of the dredgings and the atmosphere. It was concluded that vacuum drainage removed water from dredgings much faster than gravity drainage alone. A similar study using electro-osmosis rather than vacuum to dewater the dredged materials showed that the rate of drainage could be significiantly increased by that technique.

In the same report, Krizek et al. also discussed work by Dames and Moore which used mechanical equipment to agitate dredged materials in a diked disposal area to accelerate the drying process. Conditioning of the material consisted of manipulating it with a bulldozer to accelerate evaporation. It was found that the best results were obtained when thin lifts were conditioned by agitation.

The study has been repeated under more severe environmental conditions than during the earlier tests [ 19]. A two acre site at Monroe, Michigan, was covered with 18 inches of dredged

clay-water slurry. Conditioning was accomplished with two tracked vehicles for 24 hours on the first day and 10 hours a day thereafter except during rain. Although some equipment limitations were found, the areas conditioned were found to dewater much more rapidly than a control area. From an initial solids content of 29 percent, after only 8 days one of the areas had a solids content of 60 percent. In the other area the initial solids content of 21 percent increased to 72 percent after 350 coverages by the equipment. The control pond water content increased from an initial 19 percent to 35 percent. During the 30 day demonstration period about 3 inches of rain fell indicating that under dryer weather conditions even better results could have been obtained. A study by Greeley and Hansen reported on by Krizek et al. [18] showed similar results.

A study on the dewatering of dredged materials by gravity drainage has been conducted by the Philadelphia District of the Corps of Engineers. The primary purpose of the study was to provide more capacity for the storage of dredged materials within a given area. A system of parallel drainage was recommended. The ditches would be spaced 200 to 300 feet apart and would accelerate dewatering of the solids by drainage. It was estimated that a period of 2 to 10 years would be required for the area to reach a condition where agricultural equipment could be supported. However, calculations suggested that rates of consolidation could be increased by about 20 to 100 times the rate due to surcharge only.

There are several other methods of operation of diked disposal areas which could be used to increase the rate of dewatering:

. Providing a drainage system within the fill has been suggested for dewatering of power plant fly ash [20]. The objective in this

case was to maximize the amount of the waste material which could be stored in a given area.

. Filtration of water through the body of the dike has been accomplished or proposed at several Corps locations. As part of the Great Lakes pilot program the Buffalo District constructed four containment areas with pervious dikes. Two in Cleveland Harbor are stone-fill structures protected by a layer of rip-rap on the exterior face and lined on the interior with a seven foot layer of ungraded stone ranging in size from sand to large stones. The two pilot structures in Buffalo Harbor consist of slag-filled structures which filter the water as it passes through the dike. Operation of these pilot areas has been sufficiently successful to allow the Buffalo District to contract for the design and construction of full-size structures [21]. As a guide to the construction of pervious dikes, WES has published a document presenting design criteria for the selection of materials for this use [ 22 ].

. The Charleston District of the Corps has experimented with a plastic polyfilter cloth to filter dredged materials for the purpose of enabling the construction of dikes on wet ground. Although the filter material failed structurally, this and other screening methods may have merit for dewatering of dredged materials.

. The Chicago District of the Corps has propsed a diked retaining area for Milwaukee Harbor which would utilize four vertical sandfill weirs to drain water from dredged materials. The design is based on standard sanitary engineering filter criteria with the cells having three layers of graded sand and stone. It is anticipated that the filter material will have to be removed and replaced with clean material about once a year.

. The utilization of vegetation to increase the stability of

dredged material by transpiration for both confined and unconfined disposal areas is being actively studied by WES. A one year in-house study is being conducted to determine the feasibility and functional use of vegetation for slurry filtering, pollutant removal, and drying of dredged materials

A problem encountered in the design and operation of diked disposal areas is estimating the amount of dredged material which can be accommodated by a given site. As the sediment is dewatered by a combination of loading, drainage, and evapotranspiration the stored volume will decrease at a rate which depends on weather, drainage conditions, conditioning of the solids, and the nature of the solids [23]. Estimates are necessary both to relate the volume of in-place sediment to hopper dredge bin volume, and also to relate those volumes to diked area storage volume.

The first step is dredging of the bottom sediment and placement with the dredge hoppers. Numerous measurements made by the Detroit District of the Corps of Engineers have indicated the volume conversion factor may be approximately 0.82 [23]. Division of the in-place volume by 0.82 equals the hopper bin volume. The magnitude of this factor will depend on such influences as the nature of the sediment (sand, silt, or clay), whether overflowing of the hoppers during dredging is allowed, and the care with which the dredge operator performs the dredging, particularly if overflowing is not allowed.

Transfer of the hopper dredge contents to a diked disposal area will require resuspending the solids for pumping. When the solids settle out again they will occupy a volume which will decrease as consolidation occurs over a long time period. It has been reported [23] that, based on bin volume and in-place diked

area volume, a conversion factor between bin volume and disposal site volume is 0.62. This is comparable to a factor commonly used by the Corps for this purpose of 0.65. Bin volume multiplied by a factor of approximately 0.62 to 0.65 will yield disposal site volume.

During drying in the disposal site the density will be time dependent and will vary in both the horizontal and vertical directions. Krizek and Giger [23] have found that the density will increase at approximately a linear rate for at least the first ten years. From an initial density of 50 lb/cu ft soon after placement, the density will increase at a rate of 2 pounds per cubic foot per year. This will result in an increase in storage capacity of four percent per year. Of course, this process will not continue indefinitely since some final density will be approached.

A much more comprehensive discussion of diked disposal area sizing will be included in a future report by The Corps of Engineers, Waterways Experiment Station under Work Item 2C08. The report should be published late in 1975.

Other methods in addition to air drying are available for separation of water from dredged material solids. In general, these will be considerably more expensive, but situations may occur which justify their use. Pretreatment or conditioning steps include such processes as thickening, flotation, digestion, and chemical treatment. Dewatering processes include vacuum filtration, centrifugation, and other mechanical devices.

The purpose of pretreatment and/or conditioning is either to remove water, to reduce the volume, or to change the physical characteristics of the solids to make them easier to dewater. The following processes may each be applied to pretreatment of dredged material.

a. Gravity Thickening - Thickening is often the first step in removing unwanted water from sludges to reduce their volume. In many cases thickening is accomplished in the clarifier where the initial separation process occurs, but separate thickeners are also often used. Thickeners closely resemble wastewater treatment clarifiers in design and appearance, but are generally deeper and have longer residence times.

b. Flotation - The flotation process uses fine bubbles which become associated with the sludge solids, increase their buoyancy, and cause them to rise to the surface where they are removed. A variety of techniques can be used to introduce the small air bubbles used for flotation. Dispersed air flotation has been used in industrial processes, particularly in mineral separation, but it is not normally effective with light flocculent solids. Dissolved air flotation is often effective with fine-grained solids which may be organic in nature.

c. Digestion - Digestion of organic materials to change its dewatering characteristics has often been used to condition wastewater treatment plant sludges. Two different methods of digestion are available - anaerobic and aerobic. Anaerobic digestion is the decomposition of organic matter in the absence of free oxygen. Digestion occurs in a mixed culture of microorganisms where particular species are most active in different stages. In the digestion process decomposition is not complete; the products of intermediate metabolism include organic acids, ammonia, methane, hydrogen sulfide, carbon dioxide, and carbonates.

The treatment of dredged materials by anaerobic digestion has been investigated by the University of Wisconsin to provide laboratory data for a study by Nebolsine, Toth, McPhee

Associates [ 24] . Samples of dredged materials from nine Great Lakes harbors, a total of 39 samples, were seeded with digester sludge and tested in laboratory batch reactors. It was concluded that digestion was poor for all samples. This can be attributed to the fact that the solids found in dredged materials have previously existed under anaerobic conditions for extended periods, perhaps years, in the harbor sediments. Under these conditions, which are similar to those of anaerobic digestion, much decomposition will have already occurred and the volatile solids which remain will not be susceptible to further degradation by anaerobic microorganisms. The biodegradable organic matter in dredged material may also be treated by aerobic biological treatment. Other tests conducted at the University of Wisconsin on the treatability of dredged material [ 24] resulted in the conclusion that aerobic digestion did not produce any better pollutant removal than plain settling and use of the process was not justifiable.

d. Chemical Conditioning - Chemicals are commonly used to condition sludge prior to dewatering processes. Their use may be economically justified because of the increased yields and greater flexibility obtained. Chemical treatment usually involves coagulation-flocculation of sludge solids with hydrolysis products of multivalent metal ions and/or synthetic organic polymers. If polymers are used it may be necessary to test a large number of polymers and inorganic salt/polymer combinations to find an optimum conditioning combination.

e. Other Conditioning Processes - Several other conditioning processes have been used to alter the characteristics of municipal and industrial sludges. Among these are heat conditioning (Porteous Process, Farrar System, Zimpro Process), freeze conditioning, and chemical conversion (Purifax Process). At

this time it does not appear that any of these processes can be efficiently and economically applied to the conditioning of dredged material.

The objective of a dredged material mechanical dewatering system would be one or more of the following: volume reduction by separating water, alteration of properties for easier handling, or preparation for further treatment such as incineration. Vacuum filtration and centrifugation were the mechanical dewatering processes considered applicable to dredged material.

a. Vacuum Filtration - Vacuum filtration is the most commonly used mechanical sludge dewatering method in the United States. Rotary drum vacuum filters with natural or synthetic fabrics or coil springs for filtration media are used. The drum is suspended above and dips into a vat of solids. As the drum rotates slowly, part of its circumference is subject to an internal vacuum that draws solids to the filter medium. Water is drawn through the porous filter cake for that sector of the circumference. The piping arrangement within the filter permits the vacuum to be maintained until the release point where compressed air is blown through the medium to release the cake to a scraper for discharge. The filter medium may be washed before suction begins again. The function of vacuum filtration is to reduce the water content of the solids from an initial solids content of about 2 to 6 percent to about 20 to 30 percent. At that higher solids content most sludges will be a moist, easily handled cake.

The Waterways Experiment Station of the U.S. Army Corps of Engineers is conducting a study as a portion of its overall program on the disposal of dredged material [1] to determine the effectiveness and economics of vacuum filtration of dredged material. The results, which will be available in 1976, will,

to a large extent determine the ability of mechanical dewatering devices to contribute to solutions for the dredged material treatment problem.

b. Centrifugation - Although sludge centrifugation has been practiced for decades, only in recent years have centrifuges come into common use. The reasons for this increased interest are significantly improved machine design and the availability of polymers for sludge conditioning.

A wide variety of centrifuges have been developed for different applications in the process industries. These centrifuges are utilized according to their operating characteristics which best fit the process needs of washing, dewatering, classification, and clarification, or a combination of these factors. The general types of revolving container are termed basket, disc, and solid bowl. The solid bowl centrifuge is the most widely used centrifuge for sludge dewatering and contains some of the best features of other types of centrifuges. Some of its typical applications are: drilling mud classification, coal dewatering, antibiotic clarification, lime mud classification, dewatering of pulp and paper mill wastes, and dewatering sewage sludge.

Following separation of most of the water from the dredged material solids, it may be beneficial to incinerate the dredged material. Benefits would include destruction of organic pollutants such as chlorinated hydrocarbon, removal of the remaining water to minimize the required storage volume, and alteration of physical characteristics to allow easier handling. These changes would be particularly important if ultimate disposal were by landfilling at an inland site such as a quarry or for erosion control where hauling would be a large portion of the cost of the total disposal system. A related process which also requires consideration

is the production of beneficial products by kiln drying.

The most important sludge characteristics affecting the incineration process are moisture, volatiles, inerts, and calorific value. Of these, moisture is the principal factor which can be controlled to some extent by system design and operation. Moisture is generally reduced before incineration by mechanical dewatering devices to minimize the thermal load imposed on the incineration to evaporate water. It may be possible that by dewatering the solids self-sustaining combustion can be attained. The volatile percentage, and therefore the BTU value, can vary widely, so incineration equipment must be designed to handle a broad range of values.

Laboratory work for the report by Nebolsine, Toth, McPhee Associates on treatment processes for dredged materials [24] found that, although the samples selected for evaluation contained high volatile solids contents, and thus, presumably high heating values, it was not possible to ignite them even in an oven dry condition. The heating value tests in the NTMA report were performed on the entire dredged material sample. Since incineration is an expensive process it is unlikely that all dredged material solids would be put through the incineration process. A more likely system would involve separation of the inert fractions with the resulting increase in heating value per pound.

One of the most popular types of incinerators is the multiple hearth furnace because it combines simplicity and durability, and has the flexibility to burn a wide variety of materials with fluctuations in the feed rate. It consists of a circular steel shell surrounding a number of solid refractory hearths and a central slowly rotating shaft to which rabble arms are attached. Each hearth has openings to allow the solids to drop to the next lower

hearth. As the solids pass through the incinerator at various levels the solids are dried, volatile gases are driven off and burned, carbon compounds are combusted, and the residual solids are cooled.

A more recent development is the fluidized bed incinerator in which the sludge particles are fed into a bed of fluidized sand supported by upward moving air. Sufficient air is used to keep the sand in suspension, but not to carry it out of the reactor. The sand serves as a large heat reservoir where rapid mixing of the sludge throughout the bed provides efficient contact between sludge particles and oxygen, and allows rapid heat transfer. Intense and violent mixing of solids and gases in the bed which behaves as a fluid results in uniform temperature, composition, and particle size distribution throughout the bed.

A number of possible products might be made from dredged material including synthetic aggregate, lime, bricks, and mineral or rock wool. A study by Roy Weston, Inc. on Baltimore, Maryland, sediments has concluded that based on technical feasibility and market potential the only viable product for that harbor was synthetic aggregate [25] . The product would be produced by a sintering process or in a rotary kiln similar to a cement kiln. A facility which would consume $1.1 \times 10^6$ cubic yards per year of dredged material would cost about $60,000,000 and would produce a return on investment of 12 percent from sale of the product. Since San Francisco Bay dredged material is produced at about $7.0 \times 10^6$ cubic yards per year, even a plant of this size would not be a complete answer. It appears that, given the very large capital expense, while ocean dumping and land disposal alternatives exist, large scale productive use projects will probably not be undertaken.

## D. Discussion and Summary

A major consideration in optimum utilization of land disposal areas is selection of the method for drying of the solids. The most easily operated, and probably the least expensive dewatering process would be settling ponds. When dredged materials are allowed to settle for a period of several days, almost all solids will settle out. The supernatant water can be drained off and discharged to the waterway. Techniques can then be employed to further dewater the solids by air drying and drainage with the result being a dry material which can be excavated and either used/offsite as a landfill material or other productive use, or it may be stored in the disposal site. In its dryer condition, more dredged material can be stored in a given site due both to separation of a large volume of water and also to storage in greater depth above the original dike wall if the water content is sufficiently low and erosion control is practiced. The method of ultimate disposal would determine the optimum degree of dewatering.

While the dry solids from the settling pond-mechanical agitation drying system may have value as a landfill material, the most likely reusable resource in dredged material is sand and gravel if sufficient quantities and markets are available. Hydrocyclones have been found to be effective in separating sand and gravel from dredged materials. A system of hydrocyclones could operated directly on the dredged material as it enters the rehandling area via hydraulic pipeline, or the material could be temporarily stored in a diked area and later pumped through the hydrocyclone. The first case would require less equipment, but would need a higher throughput capacity. Temporary storage with rehandling may well present a better operation since it would not be directly tied to the dredging operation.

The capital cost for a 10,000 gpm hydrocyclone system for separation of sand and gravel by rehandling the material would be about $210,000. If this were depreciated over 10 years, the annual cost would be $21,000. If the annual operation and maintenance costs were 50 percent of the installed cost, or $105,000, then the total annual cost would be $126,000. Assuming that the dredged material contains 50 percent solids, 25 percent of those solids are separable, and the total annual dredging is 1 million cubic yards per year, then the volume of solids which could be separated would be 125,000 cubic yards per year, or the cost would be $1.01 per cubic yards. The value of this separation would depend on the market value of the material and cost savings associated with reduced requirements for ultimate disposal. If this were a permanent disposal area rather than a rehandling area, some cost saving could be derived from the reduced rate of filling the area thus delaying the need to locate a new area.

Quiescent settling of dredged materials for several days will produce a supernatant water with only a very small fraction of the initial suspended solids content. However, the residual suspended solids and dissolved materials may exceed limits set for discharge into the waterway adjacent to the land disposal site. Examples of potential problems are coliform bacteria, suspended solid, metals, and phosphorus. A method which would be effective in removing these contaminants is precipitation with inorganic salts and polymers in combination. The most economical method for treatment would require only a small tank for mixing of chemicals, a slightly larger tank for flocculation, and a diked settling pond. For a 5000 gpm flow the chemical mixing tank would have a volume of about 1350 cubic feet, and the flocculation tank would be 6700 cubic feet. The settling pond would have a

minimum detention time of about 12 hours, or would require an area of about one acre if the depth were 10 feet. Eventually the pond would fill up requiring that it be replaced or dredged to restore the original volume. If the coagulant requirements were 200 mg/l alum and 2 mg/l polymer, the chemical cost would be $210,000 per year. The capital cost items would be only chemical storage tanks, flash mixing equipment, and flocculation equipment and would be small compared to the chemical cost.

Some sediments, particularly those from heavily industrialized areas, may be so contaminated that land disposal may be considered inappropriate for permanent storage due to ground water contamination or other problems. A method of treating this materials is incineration which would destroy organic material and thus reduce the concentration of volatile solids, oil and grease, organohalogens, and oxygen demand to essentially zero.

A dredged material incineration system would probably be designed to serve regional needs since the capital costs are quite high. Operation should be planned for continuous processing utilizing rehandling areas to smooth out fluctuations in dredging rates. Since one of the most important factors in incineration costs is the need for auxiliary fuel, the water content of the sludge should be consistent with self-sustaining combustion if possible.

In all likelihood incinerators are an effective and economical method for altering the chemical characteristics of dredged materials. Assuming that the solids content of the dredged material was 45 percent of which 20 percent were volatile, then the capital cost of a multiple hearth furnace incineration system with a capacity of 1,000,000 tons per year would be about $17,000,000 including installed equipment, buildings, and all other equipment for an operational facility. The operating and maintenance cost

would be about $500,000 per year. With a ten year writeoff on mechanical equipment, the cost per ton of solids processed would be $2.20. The incinerator ash would be 80 percent of the original solids content, but it would be sterile and not contain any organic pollutants, so that open water disposal may be an acceptable means of ultimate disposal. If heavy metals which remain in the ash were found to be a problem, landfill of the material is also a possibility. The total cost for a system including a rehandling-drying area ($1.00 per cubic yard, incineration ($2.20 per cubic yard), and ultimate disposal ($1.00 per cubic yard) would be about $4.20 per cubic yard. While this represents a large increase over present disposal costs, in cases where highly polluted materials are encountered, the incineration system suggested may be the only viable means for disposal of these materials.

It should be emphasized that incineration cost estimates are very sensitive to the type of material. Further studies would have to be conducted to determine the heating value of dredged materials, the water content required for economical incinerator operation, and the behavior of dredged materials through the incineration process. With these studies more exact estimates of capital, operating, and maintenance costs can be derived.

## E. Conclusions

1. Regulations set by The California Regional Water Quality Control Board on turbidity and settleable matter from dredged material containment areas can generally be met by providing adequate residence time in holding ponds. The possible release of heavy metals in the diked area effluent depends on many factors which make effluent quality predictions difficult, but it appears that a well designed settling area will produce an effluent meeting realistic water quality requirements. However, in cases where

specific effluent requirements are not being met, then a coagulation treatment system could be readily and inexpensively installed to upgrade the effluent to meet the requirements. A brief study of the Mare Island land disposal site for dredged material indicated that metals were not being leached from the disposal area.

2. A number of techniques exist to facilitate the dewatering process in areas. Drying of dredged material in containment areas can be substantially hastened by agitation or by provisions to increase drainage. Mechanical dewatering of dredged material is technically feasible, but probably not economical for San Francisco Bay sediments.

3. Productive utilization of dredged material, such as manufacturing of brick or lightweight aggregate appears to be too expensive while other storage and/or disposal means are available. However, in cases where considerable amounts of sand and gravel are to be dredged, mechanical equipment is available which can readily separate these materials, but the cost-effectiveness has yet to be evaluated.

### F. Recommendations

Treatment of dredged material containment area return flows should be by gravity settling in holding ponds. If settling alone is found to be inadequate, a chemical coagulation treatment system should be installed. Consideration should be given to hastening the drying process in diked areas by providing means for drainage of water from the solids. In this way, more dredged material can be stored in a given volume and sediment stability can be increased.

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# APPENDIX A

## Miscellaneous Supporting Data

This Appendix presents representative data in support of Section III of this report. Tables A-1, A-2 and A-3 present the results of the water content and dry density tests performed on various samples from the dredges and on the diver samples. The water contents of the 50 cc syringe samples (Table A-1) ranged from 98 to 210 percent. Densities were calculated using a sample volume of 50 cc, assuming a full syringe for each sample. No sample volumes were available for the "BU" core samples (Table A-2). The water contents listed in Table A-2 were calculated in accordance with ASTM D2216-71, and dry densities were inferred from related data (see Figure 3-51).

Table A-4 presents data on COD and IOD measurements made in situ, in the BOSTON clamshell and barges filled by the BOSTON, and in the HARDING hopper dredge.

Table A-5 presents data on dissolved oxygen measurements around the clamshell dredge, in the barge, and in the hopper dredge.

Table A-6 presents data on the MTS static and vibrating cylinder tests.

Table A-1

Physical Properties of 50cc Syringe Samples

| Sample No. | Depth (ft) | Dredge | Type of Sample | Water Content (% Weight of Solids) | Dry Density * (pcf) |
|---|---|---|---|---|---|
| 117 | 32 | HARDING | 50 cc Syringe | 161.4 | 28.6 |
| 118 | 26 | | | 160.3 | 34.0 |
| 119 | 32 | | | 146.4 | 32.5 |
| 120 | 26 | | | 144.3 | 38.6 |
| 126 | 32 | | | 106.6 | -- |
| 127 | 26 | | | 137.8 | 33.4 |
| 133 | 20 | | | 177.1 | 30.0 |
| 135 | 32 | | | 111.7 | 35.3 |
| 136 | 26 | | | 139.7 | 39.1 |
| 137 | 20 | | | 144.7 | 39.3 |
| 138 | 14 | | | 210.4 | 30.0 |
| 146 | 20 | | | 175.6 | 34.8 |
| 147 | 14 | HARDING | | 197.9 | 32.4 |
| 157 | 6 | BOSTON | | 125.7 | 44.7 |
| 158 | 3 | | | 109.4 | 48.4 |
| 159 | 9 | | | 113.3 | 48.4 |
| 171 | 15 | | | 105.4 | -- |
| 172 | 12 | | | 104.1 | 47.6 |
| 173 | 9 | | | 122.9 | 45.8 |
| 174 | 6 | | | 98.2 | -- |
| 175 | 3 | BOSTON | 50cc Syringe | 133.6 | 37.0 |

*All densities were calculated based on the assumption that the volume of the samples was 50cc, i.e. the syringe was full before sample was put into plastic containers.

Table A-2

Physical Properties of "BU" Core Samples

| Sample No. | Dredge | Type of Sample | Water Content (% Weight of Solids) | Inferred Density* (pcf) |
|---|---|---|---|---|
| 01 | BOSTON | BU CORE | 141.9 | 34 |
| 02 | | | 105.2 | 44 |
| 03 | | | 116.1 | 41 |
| 045 | | | 194.4 | 22 |
| 046 | | | 132.8 | 36 |
| 047 | | | 171.1 | 27 |
| 048 | | | 154.2 | 31 |
| 049 | | | 167.7 | 28 |
| 050 | | | 120.2 | 40 |
| 051 | | | 153.5 | 31 |
| 054 | | | 120.2 | 40 |
| 055 | | | 111.5 | 42 |
| 056 | | | 103.5 | 45 |
| 057 | | | 177.0 | 26 |
| 058 | | | 99.2 | 46 |
| 154 | | | 90.5 | 49 |
| 155 | | | 57.5 | 65 |
| 156 | BOSTON | BU CORE | 113.0 | 42 |

*Dry densities were taken off plot of dry densities vs. water content using average curve for barrel samples (See Figure 3-51).

Table A-3

Physical Properties of Diver Samples

| Sample No. | Type of Sample | Water Content (% weight of Solids) | Dry Density * (pcf) |
|---|---|---|---|
| 180 | DIVER | 251.4 | 21.7 |
| 185 Top | ↑ | 161.4 | 32.5 |
| 185 Bottom | | 117.9 | 39.7 |
| 190 | | 157.2 | 30.8 |
| 192 | | 135.5 | 36.7 |
| 194 | | 144.7 | 35.6 |
| 196 | | 212.5 | 24.9 |
| 203 | | 141.1 | 35.5 |
| 204 Top | | 147.7 | 35.1 |
| 204 Bottom | | 148.7 | 34.6 |
| 206 | | 156.6 | 31.8 |
| 209 | | 142.1 | 32.7 |
| 211 | | 146.5 | 34.3 |
| 213 | | 182.2 | 27.5 |
| 228 Top | | 172.8 | 30.3 |
| 228 Bottom | ↓ | 123.6 | 36.5 |
| 229 | DIVER | 153.3 | 32.5 |

*Sample volumes were determined by marking plastic containers (top of sample), and measuring volume of water used to replace sample.

Table A-4

COD and IOD Data

| Date/ Time | Sample Number | COD mg/kg | IOD mg/kg | Remarks |
|---|---|---|---|---|
| 3/11 | | | 645 | Clamshell bucket (run 4 hours after sampling) |
| 1010 | | | 889 | Pocket #5, 10' deep |
| 1010 | | | 616 | Pocket #5, 5' deep |
| 1010 | | | 1195 | Pocket #5, 1.5' deep |
| 1100 | 009 | 52,600 | | Clamshell bucket |
| 1125 | 017 | 50,600 | | Partially filled #4 pocket |
| 1300 | 024 | 50,400 | | Partially filled #3 procket |
| 1600 | | | 1000 | Pocket #1, 10' deep |
| 1625 | | | 1018 | Pocket #4, 2' deep |
| 1640 | | | 862 | Pocket #4, 10' deep |
| 1700 | | | 1000 | Pocket #6, 10' deep |
| 3/12 | | | | |
| 0930 | 062 | 45,200 | | Totally disturbed surface Pocket #5 |
| 0935 | 064 | 51,500 | | Clump, pocket #5 |
| 1210 | 067 | 40,800 | | Clamshell bucket |
| 1225 | | | 891 | Clamshell bucket |
| 1232 | | | 785 | Clamshell bucket |
| 1235 | | | 1205 | Clamshell bucket |
| 3/13 | | | | |
| 0940 | | | 1000 | Max density flow, hopper #3 |
| 0940 | | | 842 | Dupe |
| 0943 | 171 | 45,000 | | Maxdensity flow, hopper #3 |
| 0943 | 072 | 49,100 | | Min density flow, hopper #3 |
| 1055 | | | 349 | Medium density flow, #3 |
| 1055 | | | 677 | Dupe |
| 1325 | 085 | 49,900 | | Pail sample, 29' deep, #3 |
| 1530 | | | 1,000 | |
| 3/15 | | | | |
| 0740 | 122 | 72,200 | | High density flow, #3 |
| 0820 | 131 | 51,300 | | Hopper #3, 7.5'depth |
| 0830 | | | 693 | Hopper #3, 7.5' depth |
| 0830 | | | 827 | Hopper #3, 7.5' depth (liquid) |
| 0945 | 148 | 51,200 | | Hopper #3, 10.5' depth |
| | 148 | | 550 | Hopper #3, 10.5' depth (soupy) |
| 0954 | 144 | | 834 | Hopper #3, 32' deep |

Table A-4 (Continued)

| Date/ Time | Sample Number | COD mg/kg | IOD mg/kg | Remarks |
|---|---|---|---|---|
| 3/17 | | | | |
| 1150 | 165 | 50,100 | | Clamshell bucket |
| 1150 | 167 | 52,800 | | Clamshell bucket |
| 1200 | 169 | 49,400 | | #2 pocket, surface material |
| 1200 | 170 | 39,500 | | #1 pocket, mud-water interface |
| 3/18 | | | | |
| 1000 | 177 | 48,800 | | In situ, Mare Island, North of dredged area (soupy) |
| | 177 | | 1220 | In situ, Mare Island, North of dredged area (soupy) |
| | 177 | | 1890 | In situ, Mare Island, North of dredged area (soupy) |
| 1000 | 178 | 49,500 | | 3' to side of #177 |
| | 187 | 51,900 | | In situ, Mare Island, dredged area |
| | 191 | 52,300 | | In situ, Mare Island (1.5' below bottom interface) |
| | 193 | 49,100 | | In situ, Mare Island 1' below interface |
| | 195 | 52,000 | | In sediment at mud/water interface |
| 3/19 | | | | |
| 1346 | 205 | 55,300 | 605, 728 | In situ, Alameda NAS, Loc. #1 |
| | 208 | 54,500 | 800 | 1.5' below bottom interface |
| | 210 | 57,000 | | 1' below bottom interface |
| | 212 | 53,200 | | bottom interface |

Table A-5

Dissolved Oxygen

| Date | Location | Sample # | Depth | (PPM) | Remarks |
|---|---|---|---|---|---|
| 3/11 | Alameda 1 | 033 | 3' | 10.4 | Background *in situ* water |
| | Alameda 1 | 032 | 20' | 10.0 | Background *in situ* water |
| | Alameda 1 | 031 | 35' | 7.0 | Background *in situ* water |
| | Alameda 1 | | 3' | 10.0 | Edge of plume |
| | Alameda 1 | | 20' | 10.1 | Edge of plume |
| | Alameda 1 | | 35' | 9.2 | Edge of plume |
| | Alameda 1 | | 3' | 10.3 | In plume, 25' downstream |
| | Alameda 1 | | 20' | 10.0 | In plume, 25' downstream |
| | Alameda 1 | | 35' | 10.0 | In plume, 25' downstream |
| | Alameda 1 | | 3' | 10.0 | In plume, 50' downstream |
| | Alameda 1 | | 20' | 9.4 | In plume, 50' downstream |
| | Alameda 1 | | 35' | 10.2 | In plume, 50' downstream |
| 3/11 | Alameda 1 | | Surface | 0.8 | Barge pocket water |
| 3/12 | Alameda 2 | | 3' | 6.6 | Barge overflow (water column) |
| | Alameda 2 | | 3' | 8.0 | *In situ* background |
| 3/17 | Alameda 3 | | 3" | 0.6 | Barge pocket water |
| | | | Surface | 1.1 | Barge pocket water |
| 3/15 | Mare Island | | Surface | 1.6 | Hopper #3 |
| | Mare Island | | 3" | 1.3 | Hopper #3 |
| | Mare Island | | 6" | 1.3 | Hopper #3 |

Table A-6

## Raw Laboratory Data for MTS Program

| SAMPLE NO. | WET WEIGHT (gms) MTS SAMPLE | WET WEIGHT (gms) STATIC SAMPLE | DRY WEIGHT (gms) MTS SAMPLE | DRY WEIGHT (gms) STATIC SAMPLE | VOLUME (cc) MTS SAMPLE | VOLUME (cc) STATIC SAMPLE |
|---|---|---|---|---|---|---|
| S-1-A | 75.29 | 76.23 | 48.74 | 49.49 | 47 | 46 |
| S-1-B | 72.06 | 75.95 | 46.65 | 49.94 | 43 | 45 |
| S-1-C | 79.03 | 76.97 | 51.25 | 50.61 | 47 | 46 |
| S-1-D | 76.77 | 78.13 | 50.14 | 50.95 | 46 | 47 |
| S-1-E | 75.67 | 76.27 | 49.33 | 49.73 | 46 | 46 |
| S-1-F | 77.00 | 79.98 | 50.49 | 51.61 | 47 | 48 |
| S-2-A | 71.37 | 74.73 | 45.16 | 48.28 | 43 | 45 |
| S-2-B | 74.48 | 74.56 | 47.82 | 47.58 | 45 | 45 |
| S-2-C | 73.01 | 74.49 | 46.64 | 47.09 | 44 | 45 |
| S-2-D | 74.52 | 73.11 | 47.90 | 46.98 | 45 | 44 |
| S-2-E | 74.61 | 74.43 | 47.91 | 47.68 | 45 | 45 |
| S-2-F | 74.56 | 74.35 | 47.69 | 47.28 | 45 | 45 |
| S-3-A | 57.46 | 58.25 | 29.38 | 29.72 | 40 | 40 |
| S-3-B | 57.11 | 57.16 | 29.26 | 29.08 | 40 | 40 |
| S-3-C | 57.19 | 58.37 | 29.22 | 29.69 | 40 | 40 |
| S-3-D | 57.27 | 59.95 | 28.89 | 30.86 | 40 | 40 |
| S-3-E | 59.01 | 58.73 | 29.93 | 30.22 | 40 | 40 |
| S-3-F | 59.00 | 57.61 | 29.61 | 29.44 | 40 | 40 |
| S-4-A | 66.60 | 63.55 | 33.33 | 32.51 | 46 | 43 |
| S-4-B | 66.58 | 64.98 | 33.48 | 33.12 | 45 | 44 |
| S-4-C | 70.49 | 67.95 | 35.93 | 34.43 | 47 | 46 |
| S-4-D | 64.99 | 61.46 | 32.25 | 30.88 | 44 | 42 |
| S-4-E | 56.41 | 55.67 | 28.16 | 27.94 | 38 | 38 |
| S-4-F | 59.82 | 55.60 | 29.76 | 27.73 | 40 | 37 |
| S-5-A | 42.21 | 45.58 | 10.19 | 9.88 | 40 | 35 |
| S-5-B | 45.62 | 47.02 | 10.19 | 10.31 | 39 | 40 |
| S-5-C | 45.65 | 46.99 | 9.91 | 11.07 | 39 | 40 |
| S-5-D | 44.13 | 44.43 | 7.81 | 8.00 | 39 | 39 |
| S-5-E | 45.52 | 45.74 | 9.94 | 10.52 | 40 | 39 |
| S-5-F | 56.09 | 56.69 | 26.61 | 28.03 | 39 | 39 |
| S-6-A | 48.03 | 46.63 | 10.31 | 9.55 | 41½ | 41½ |
| S-6-B | 46.70 | 47.71 | 9.98 | 10.27 | 40½ | 41½ |
| S-6-C | 48.86 | 47.68 | 10.20 | 10.18 | 41½ | 42 |
| S-6-D | 45.03 | 47.70 | 8.04 | 8.00 | 41 | 42½ |
| S-6-E | 46.15 | 46.54 | 9.95 | 10.11 | 40½ | 41 |
| S-6-F | 58.87 | 57.05 | 29.58 | 26.25 | 41 | 41 |
| S-7-A | 49.33 | 50.72 | 14.10 | 14.77 | 41 | 42 |
| S-7-B | 48.36 | 49.57 | 14.10 | 14.14 | 40 | 41 |
| S-7-C | 49.23 | 51.71 | 14.07 | 14.82 | 41 | 43 |
| S-7-D | 50.04 | 48.41 | 13.90 | 12.79 | 42 | 41 |
| S-7-E | 49.71 | 47.46 | 13.15 | 12.63 | 42 | 40 |
| S-7-F | 52.68 | 53.56 | 16.50 | 16.21 | 43 | 44 |

Table A-6 (cont.)

| | | | | | | |
|---|---|---|---|---|---|---|
| C-14-A | 45.43 | 48.66 | 5.52 | 5.93 | 41 | 44 |
| C-14-B | 46.99 | 47.70 | 5.68 | 5.83 | 43 | 43 |
| C-14-C | 48.03 | 46.87 | 5.77 | 5.61 | 44 | 42 |
| C-14-D | 47.74 | 48.25 | 8.55 | 8.41 | 43 | 44 |
| C-14-E | 51.08 | 47.99 | 10.34 | 9.81 | 45 | 42 |
| C-14-F | 49.53 | 50.23 | 10.99 | 11.48 | 43 | 43 |
| C-15-A | 46.08 | 46.10 | 9.45 | 9.75 | 41 | 41 |
| C-15-B | 45.24 | 45.42 | 9.40 | 9.12 | 40 | 40 |
| C-15-C | 44.98 | 46.03 | 9.22 | 9.30 | 40 | 41 |
| C-15-D | 46.47 | 46.05 | 9.06 | 9.10 | 41 | 41 |
| C-15-E | 48.29 | 47.01 | 9.94 | 9.22 | 42 | 42 |
| C-15-F | 47.73 | 47.44 | 10.16 | 9.85 | 41 | 42 |
| C-16-A | 46.39 | 45.71 | 9.60 | 9.47 | 41 | 41 |
| C-16-B | 45.45 | 45.29 | 9.57 | 9.46 | 40 | 40 |
| C-16-C | 45.18 | 46.20 | 9.23 | 9.58 | 40 | 41 |
| C-16-D | 48.41 | 47.44 | 10.10 | 9.10 | 43 | 43 |
| C-16-E | 48.60 | 46.12 | 10.25 | 9.43 | 43½ | 41½ |
| C-16-F | 50.24 | 48.63 | 11.38 | 10.39 | 43½ | 43 |
| C-18-A | 56.31 | 57.12 | 21.13 | 21.53 | 42 | 42 |
| C-18-B | 54.66 | 55.41 | 20.00 | 20.58 | 42½ | 42 |
| C-18-C | 57.62 | 55.96 | 21.34 | 21.25 | 42 | 43 |
| C-18-D | 55.75 | 54.05 | 20.13 | 19.80 | 44 | 42 |
| C-18-E | 58.41 | 58.25 | 21.76 | 22.69 | 45 | 45 |
| C-18-F | 54.03 | 58.24 | 20.40 | 22.57 | 42 | 45 |

Table A-6 (cont.)

| | | | | | | |
|---|---|---|---|---|---|---|
| S-8-A | 55.04 | 55.04 | 15.25 | 15.33 | 45 | 45 |
| S-8-B | 55.34 | 55.62 | 15.36 | 15.50 | 45 | 45 |
| S-8-C | 55.03 | 55.56 | 15.20 | 15.50 | 45 | 45 |
| S-8-D | 55.87 | 55.49 | 15.25 | 14.67 | 45 | 45 |
| S-8-E | 55.76 | 54.44 | 15.40 | 14.98 | 45 | 45 |
| S-8-F | 55.78 | 56.67 | 17.09 | 16.57 | 45 | 45 |
| S-17-A | 64.48 | 66.16 | 30.82 | 31.18 | 45 | 46 |
| S-17-B | 66.61 | 65.41 | 31.85 | 30.68 | 46 | 46 |
| S-17-C | 64.36 | 66.58 | 30.49 | 31.35 | 46 | 46 |
| S-17-D | 67.32 | 67.86 | 33.12 | 33.50 | 46 | 47 |
| S-17-E | 67.55 | 66.29 | 31.57 | 31.67 | 46 | 46 |
| S-17-F | 66.24 | 67.44 | 32.13 | 31.59 | 45 | 46 |
| C-9-A | 62.99 | 56.68 | 33.81 | 31.01 | 43 | 39½ |
| C-9-B | 67.30 | 66.52 | 36.04 | 36.66 | 45 | 45 |
| C-9-C | 66.56 | 64.69 | 35.43 | 35.69 | 45 | 44½ |
| C-9-D | 66.06 | 66.02 | 35.25 | 35.97 | 44 | 44½ |
| C-9-E | 65.01 | 61.07 | 34.74 | 33.32 | 43 | 41 |
| C-9-F | 64.70 | 63.17 | 34.85 | 34.25 | 43 | 42 |
| C-10-A | 66.59 | 69.07 | 34.99 | 35.93 | 45 | 46 |
| C-10-B | 69.48 | 68.22 | 36.96 | 35.39 | 46 | 46 |
| C-10-C | 69.64 | 69.09 | 37.14 | 36.24 | 46 | 46 |
| C-10-D | 70.74 | 68.42 | 37.42 | 35.51 | 47 | 45 |
| C-10-E | 69.39 | 67.53 | 36.15 | 36.33 | 46 | 45 |
| C-10-F | 69.75 | 69.92 | 36.41 | 37.10 | 46 | 46 |
| C-11-A | 55.45 | 50.55 | 23.69 | 21.10 | 42 | 38 |
| C-11-B | 62.02 | 64.18 | 25.55 | 26.47 | 45 | 46 |
| C-11-C | 61.24 | 61.22 | 25.30 | 25.38 | 44 | 45 |
| C-11-D | 56.20 | 39.82 | 21.93 | 15.95 | 43 | 30 |
| C-11-E | 53.43 | 42.87 | 19.49 | 15.31 | 41 | 33 |
| C-11-F | 45.04 | 37.70 | 16.57 | 11.58 | 34 | 30 |
| C-12-A | 53.71 | 54.05 | 20.80 | 20.74 | 42 | 41½ |
| C-12-B | 52.39 | 52.83 | 20.43 | 20.52 | 40 | 40½ |
| C-12-C | 54.02 | 53.68 | 20.97 | 20.78 | 41 | 41 |
| C-12-D | 53.25 | 51.40 | 19.84 | 19.80 | 41½ | 40 |
| C-12-E | 52.97 | 51.52 | 20.20 | 19.65 | 41 | 40 |
| C-12-F | 53.55 | 51.65 | 19.87 | 19.67 | 42 | 40 |
| C-13-A | 48.90 | 44.94 | 9.04 | 7.91 | 44 | 40 |
| C-13-B | 51.21 | 46.22 | 9.39 | 8.59 | 45 | 40 |
| C-13-C | 46.29 | 47.39 | 10.21 | 8.54 | 39 | 42 |
| C-13-D | 43.56 | 47.60 | 7.99 | 7.54 | 39 | 43 |
| C-13-E | 50.08 | 47.83 | 12.23 | 9.88 | 43 | 41 |
| C-13-F | 56.26 | 56.97 | 25.89 | 23.94 | 40 | 42 |

## APPENDIX B

### Shear Vane Data

### Rod Friction

The values shown on Figure B-1 represent the results obtained when the drilling rods were rotated with a torque wrench while the shear vane was not attached. As time was severely limited in the field, only three such tests could be performed. A rotation through 60° was achieved in all tests at the rate of 15 degrees per minute. The shear stress was calculated as if the shear vane were attached and the results were plotted. A numerical average of the three measured shear strengths was then taken at 15° rotation intervals, and the resulting dashed curve was taken to represent the rod friction values for the average depth of eight feet. The points on this curve could then be normalized to the solid curve for one-foot rod sections by dividing the shear strength values by eight. Since the results were fairly constant after 10° of rotation, a value of 1.25 psf of rod friction per foot of rod was adopted for all applicable calculations.

The correcting factor of 1.25 psf per foot of rod was then applied for all tests on the basis of the length of rod penetration into materials more consistent than water, i.e., the depth to resistance averaged for each day, computed from the penetration test data. On the HARDING these depths ranged from 22 to 24 feet. The results of any tests deeper than that were accordingly corrected. On the BOSTON the rod friction corrections were applied to the respective test depths, since there the actual rod penetration into cohesive material was known.

### Shear Vane Calibration

The shear strength determined using shear vanes was calculated in accordance with ASTM D 2573-72 "Standard Method for Field Vane Shear Test in Cohesive Soil." The shear strength equals the torque divided by a constant, K. K is determined by the formula given in the ASTM standards and is only dependent upon the dimensions of the shear vane. If the torque is recorded in in-lbs, the vane dimensions would be measured in inches. K would be in cubic inches, and the shear strength would be calculated in pounds per square inch.

### Data

Figures B-2 through B-18 show shear stress measurements made on the HARDING and in barges filled by the BOSTON.

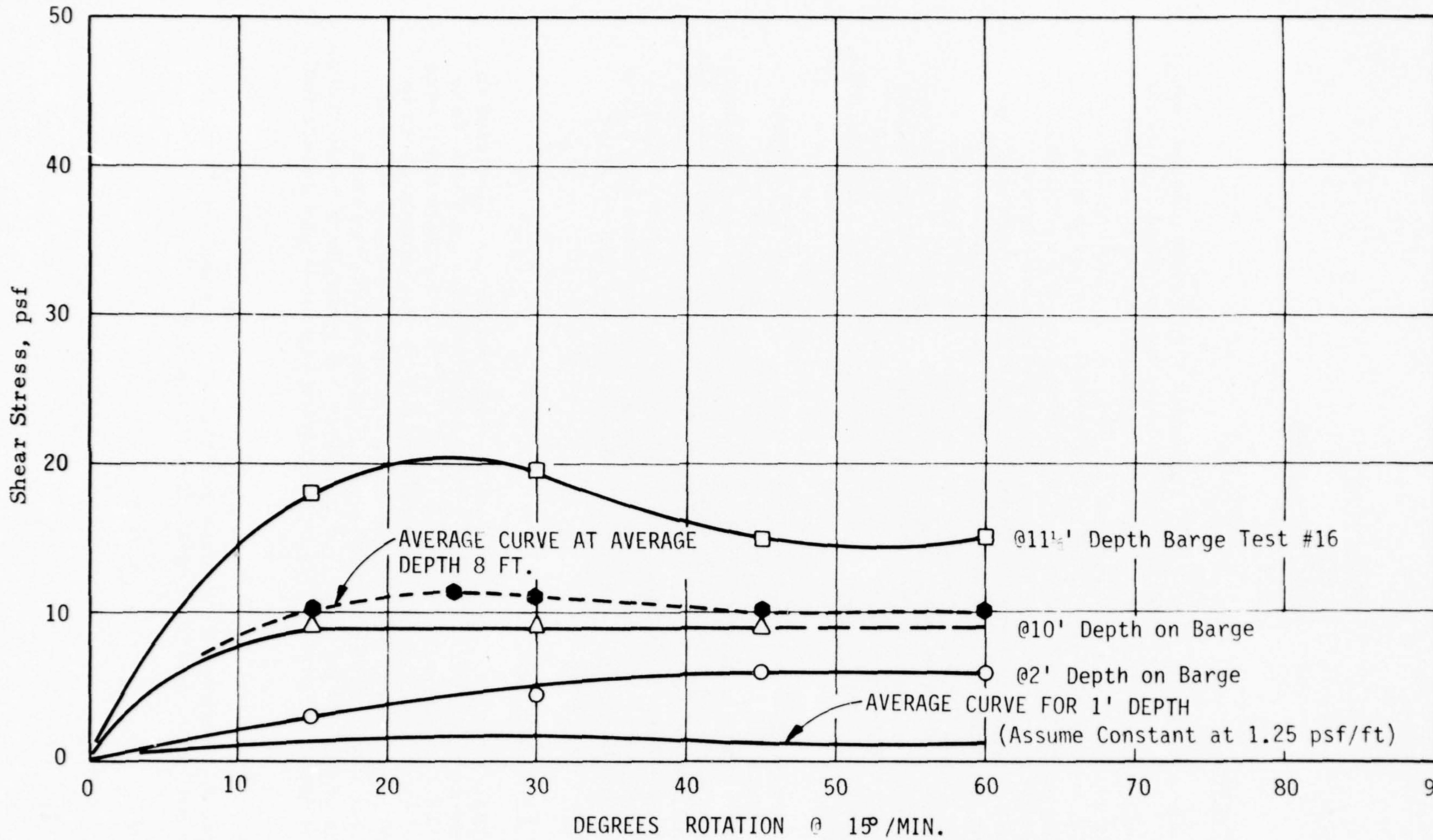


Figure B-1. Rod Friction Data for Field Vane Shear Tests

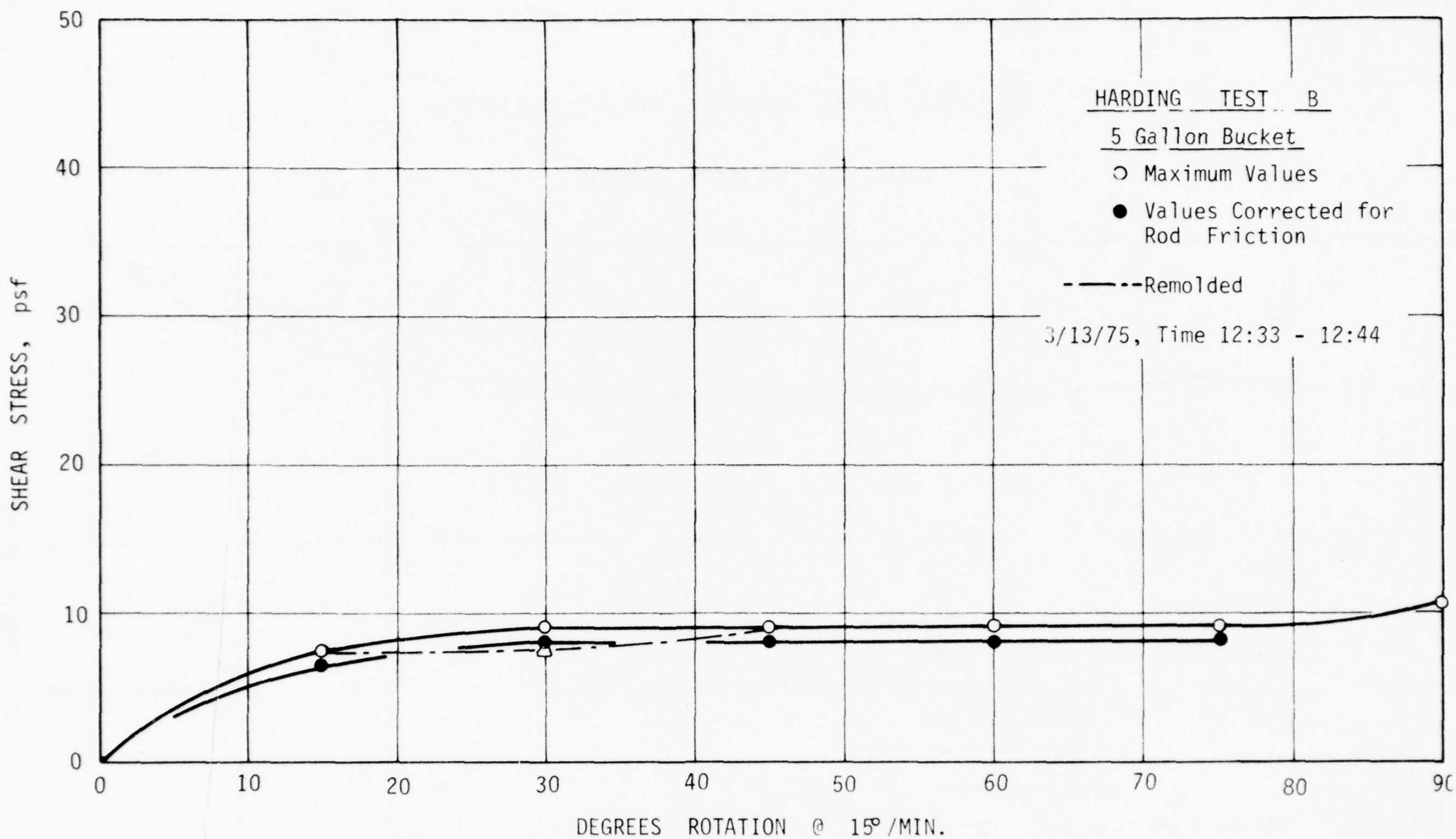


Figure B-2. Field Vane Shear Tests - HARDING

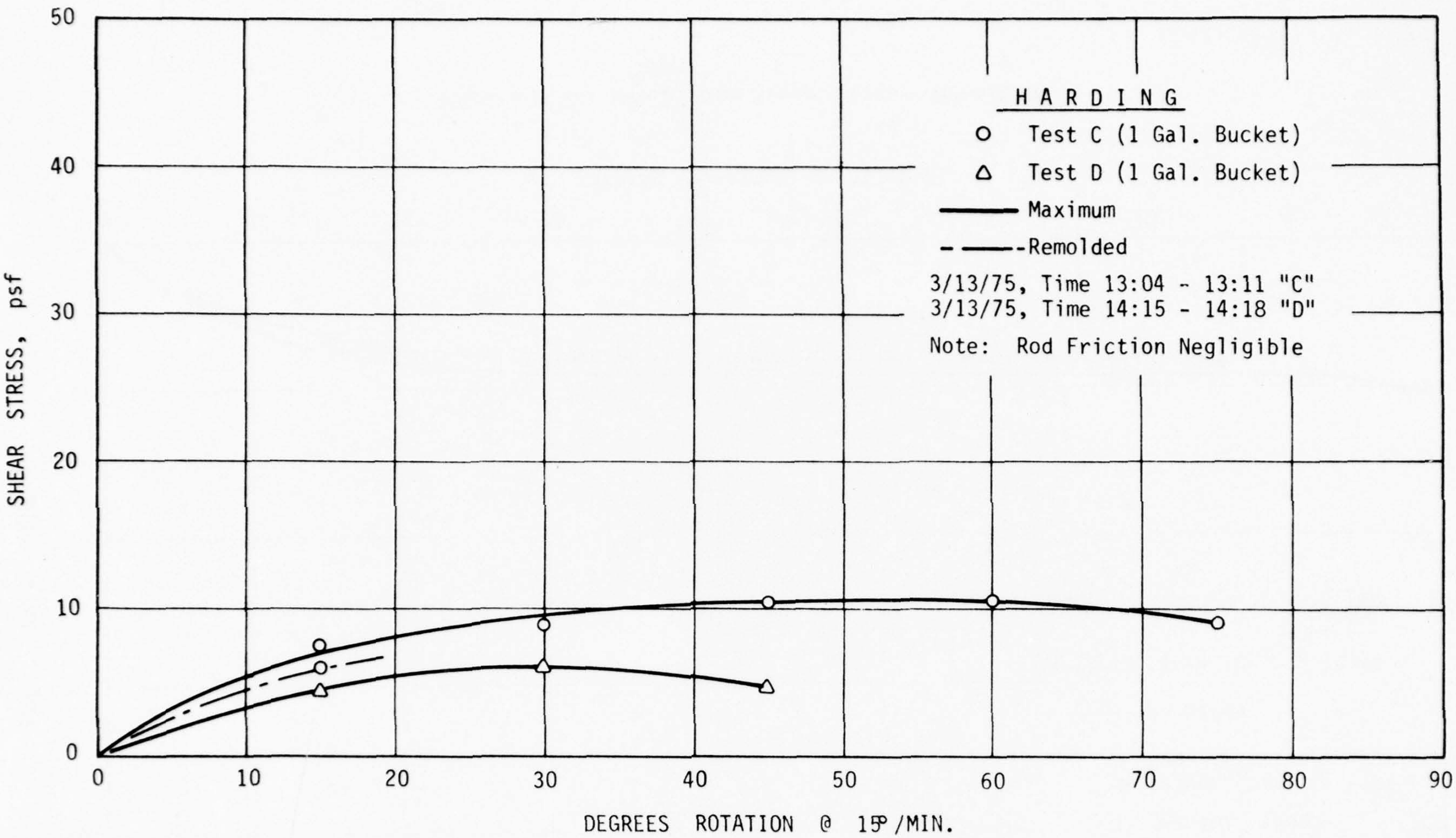


Figure B-3. Field Vane Shear Tests - HARDING

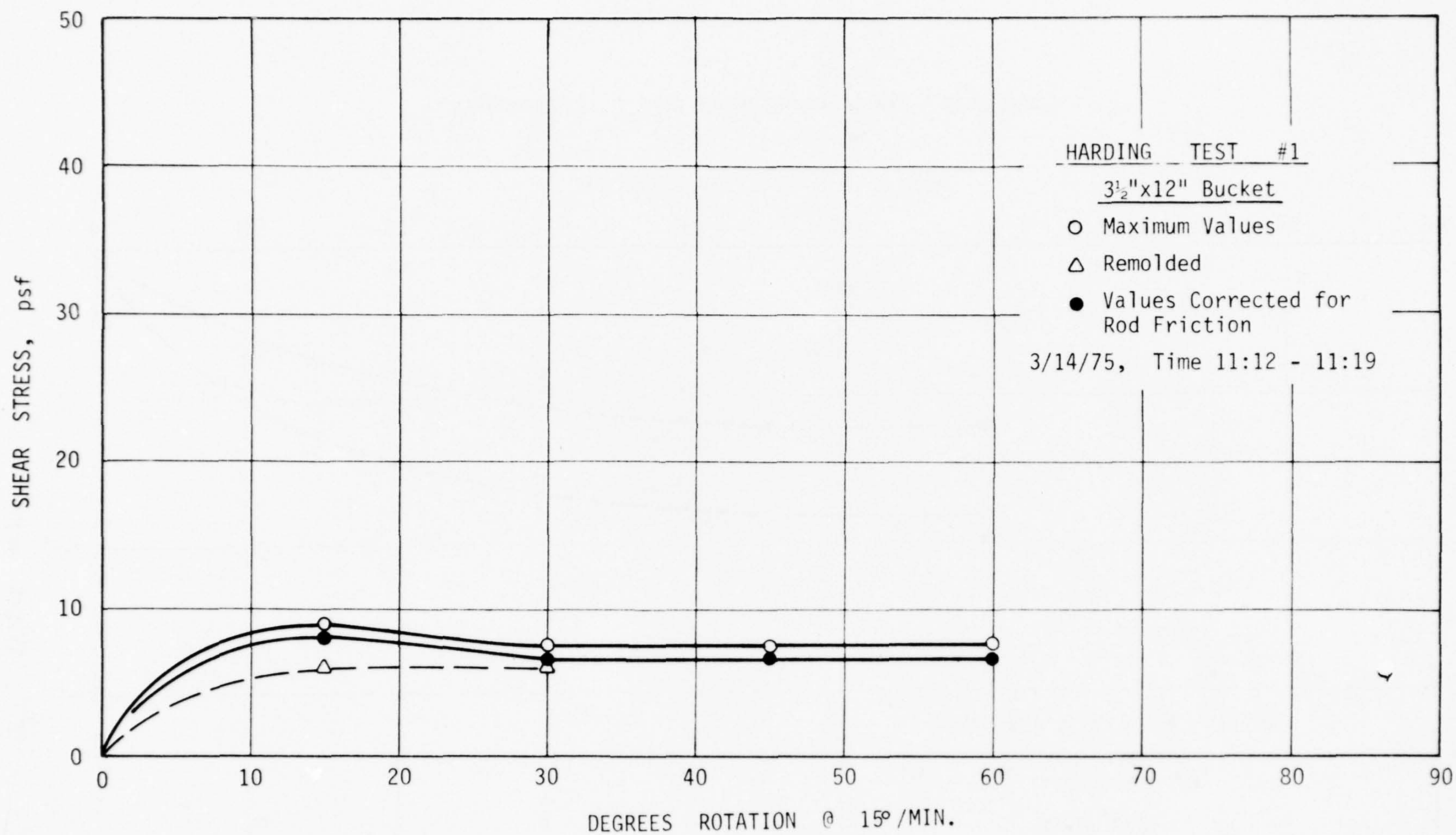


Figure B-4. Field Vane Shear Tests - HARDING

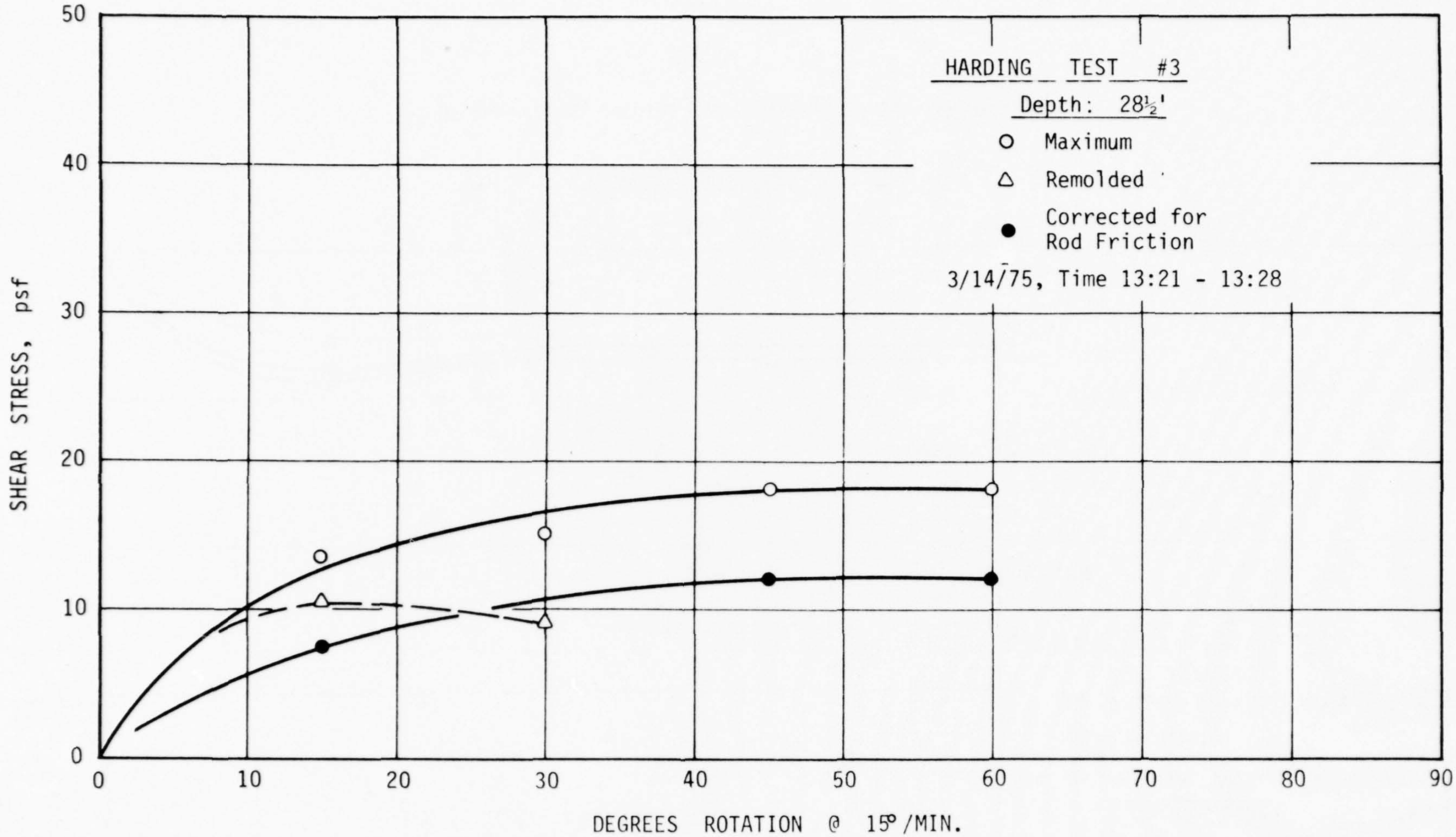


Figure B-5. Field Vane Shear Tests - HARDING

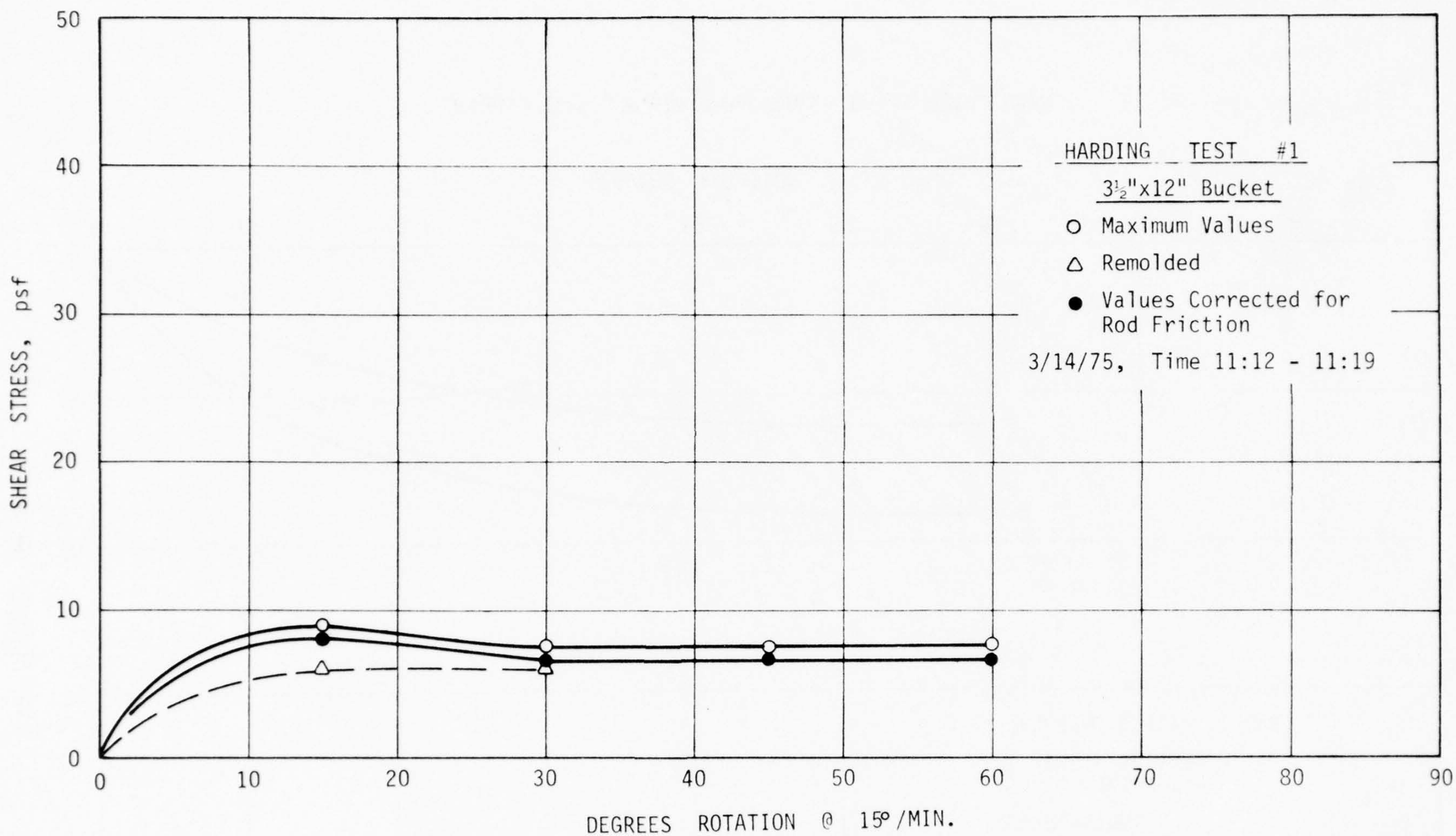


Figure B-4. Field Vane Shear Tests - HARDING

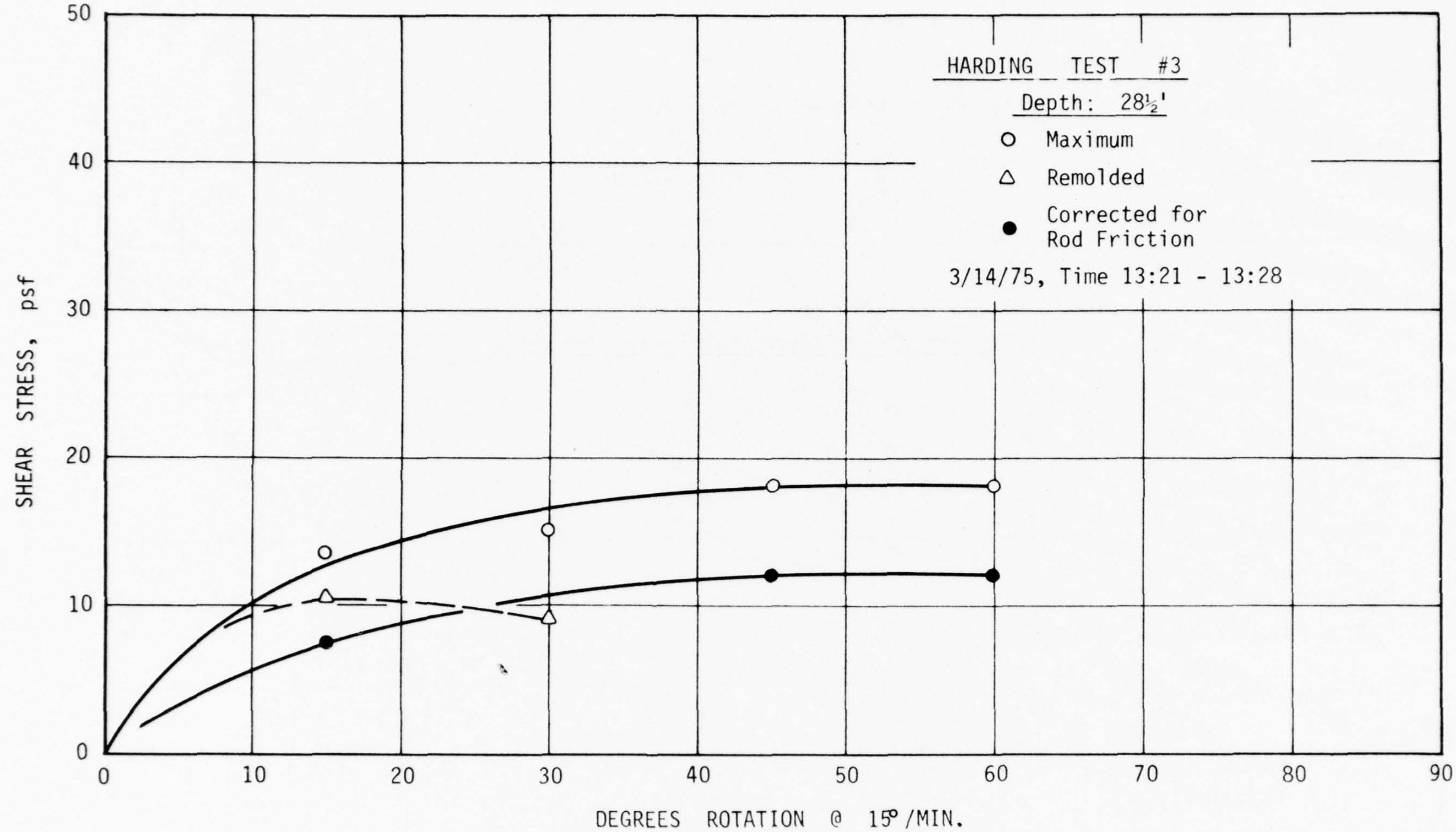


Figure B-5. Field Vane Shear Tests - HARDING

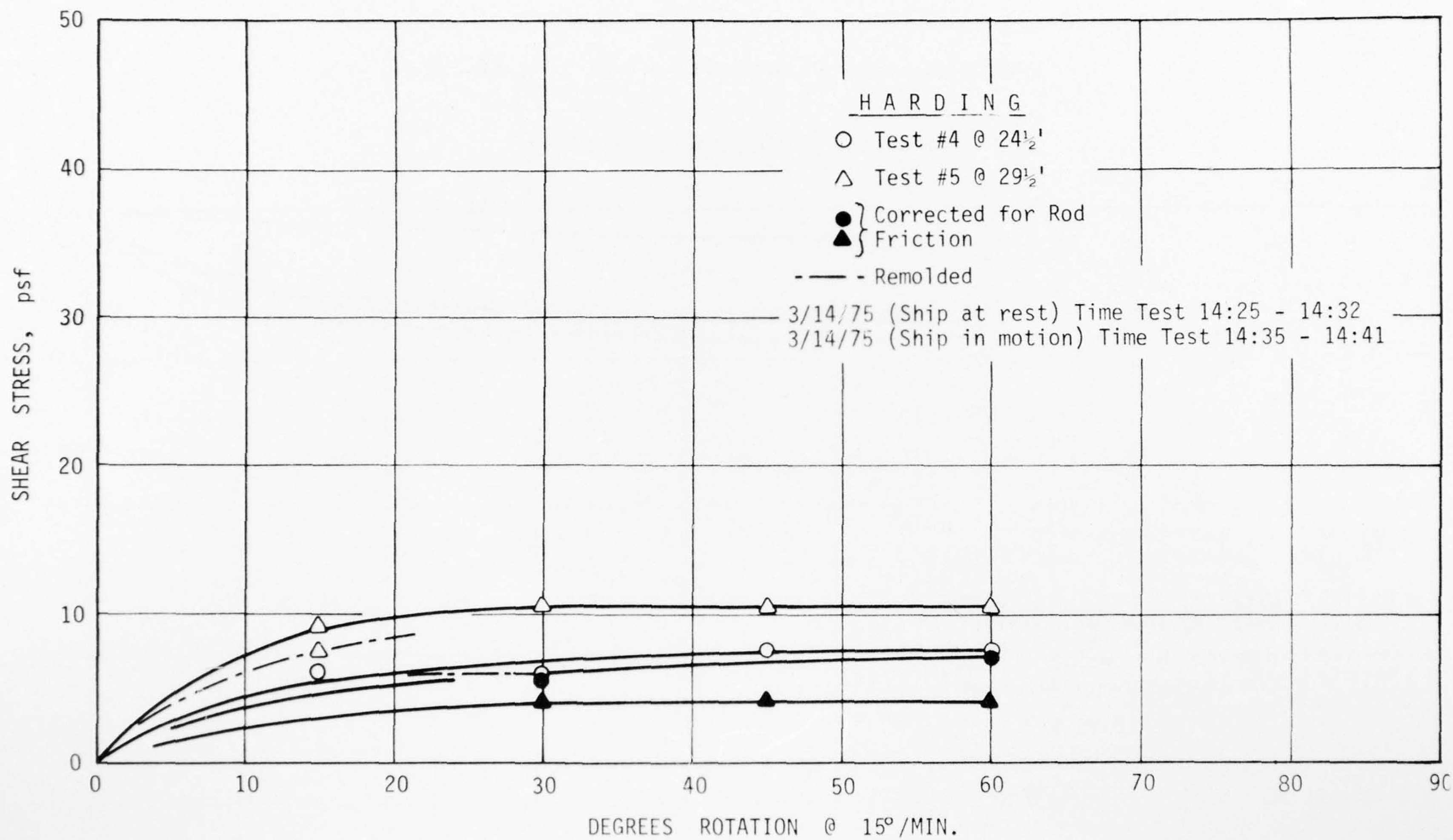


Figure B-6. Field Vane Shear Tests - HARDING

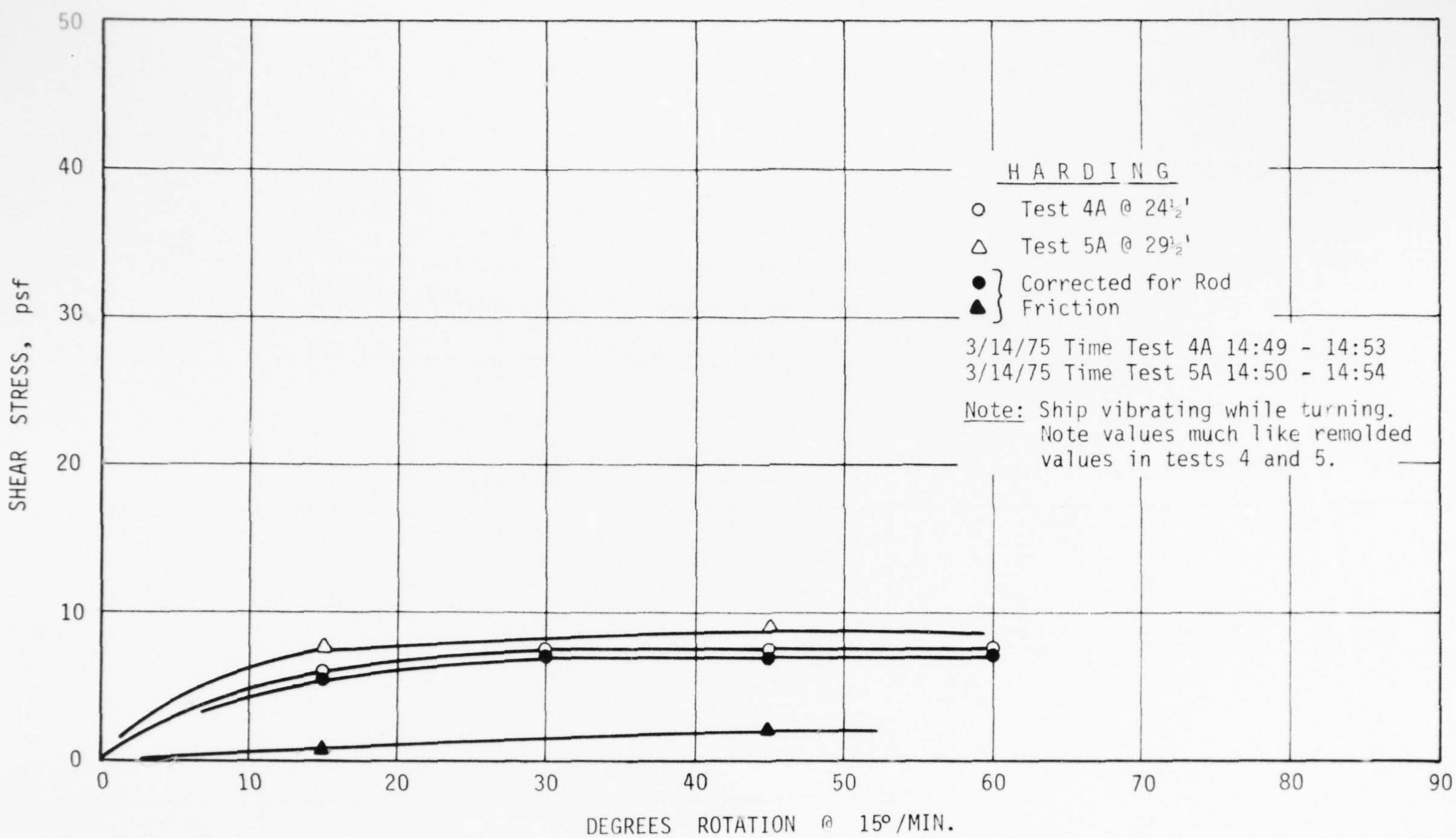


Figure B-7. Field Vane Shear Tests - HARDING

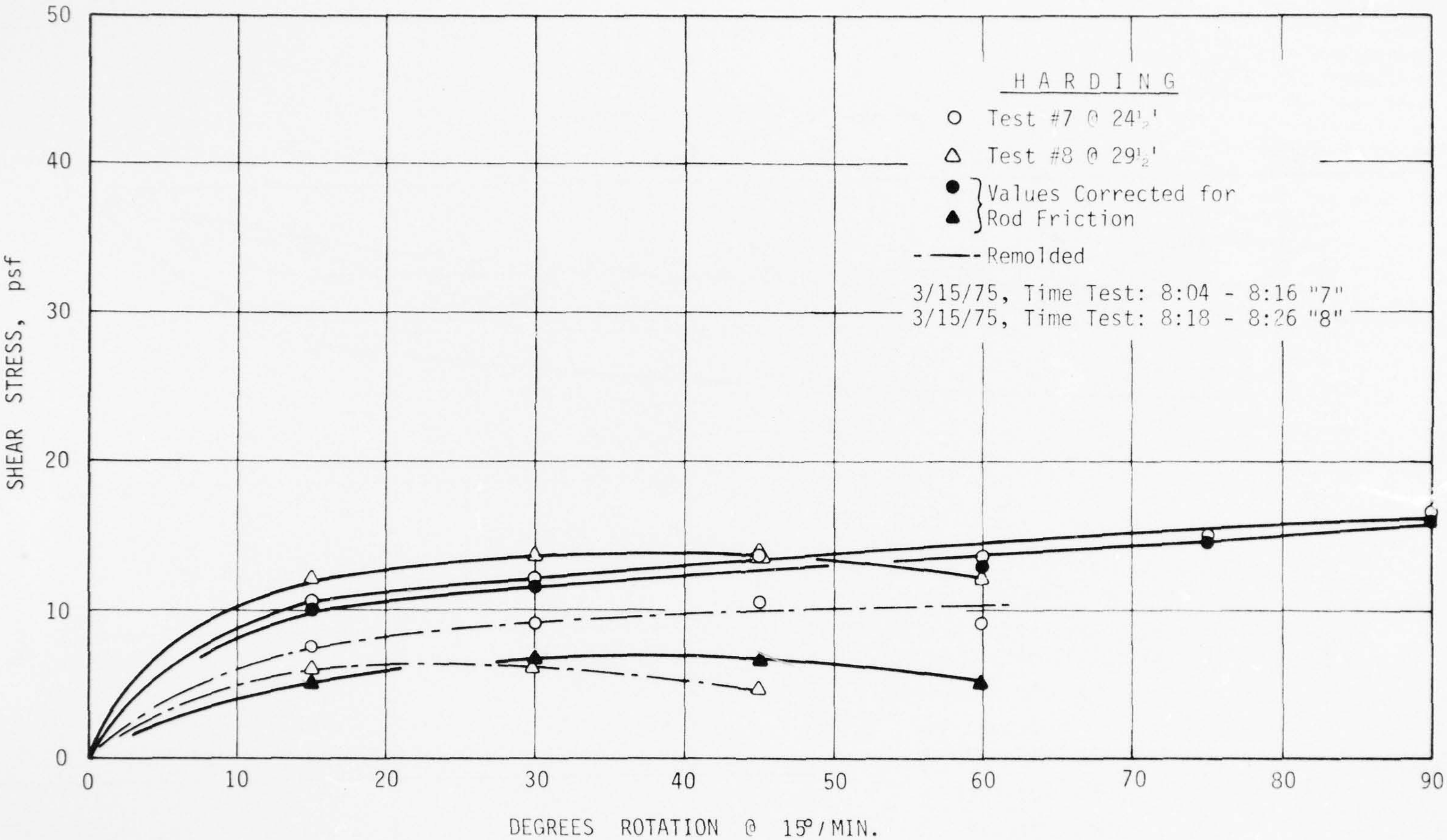


Figure B-8. Field Vane Shear Tests - HARDING

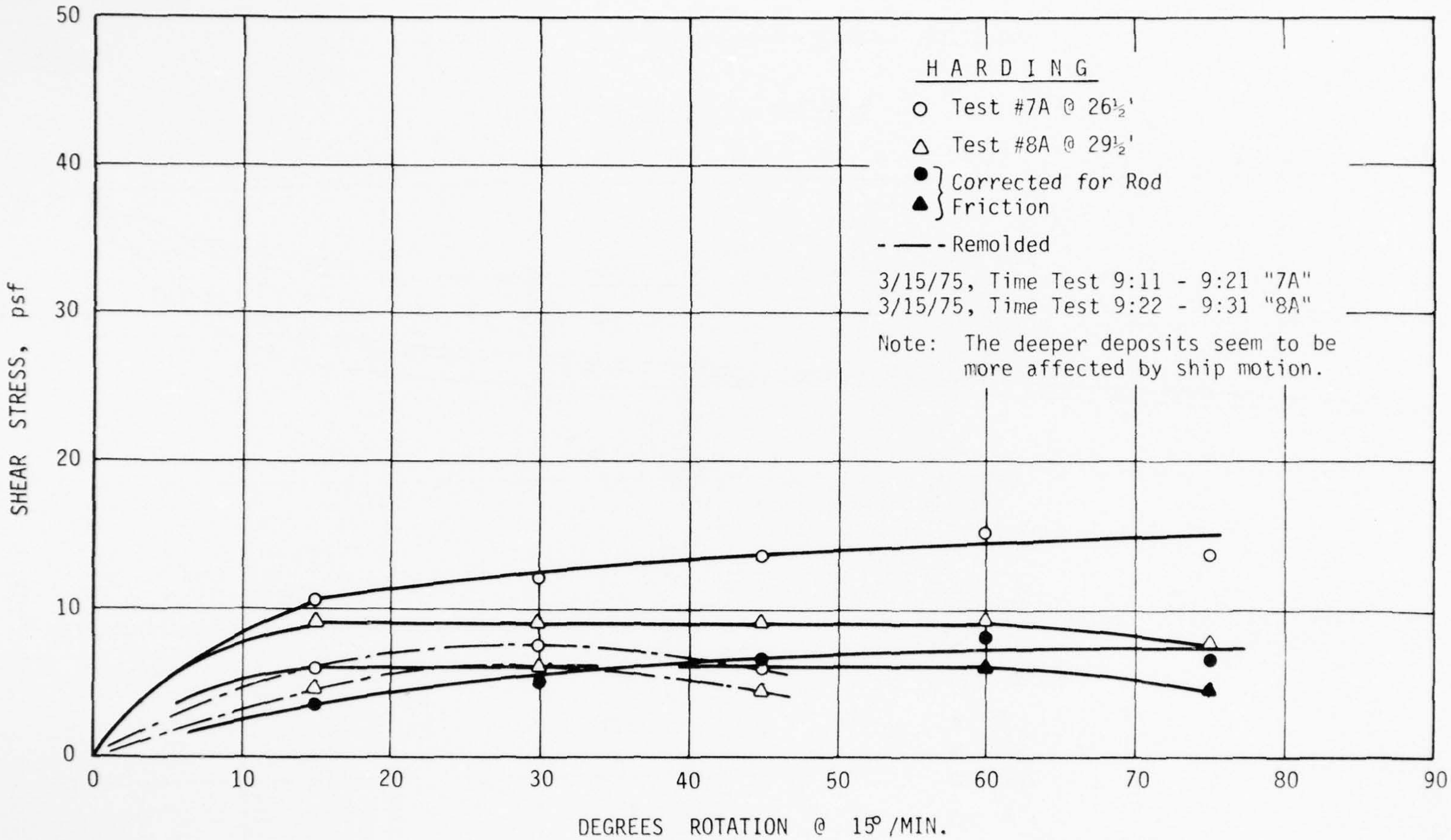


Figure B-9. Field Vane Shear Tests - HARDING

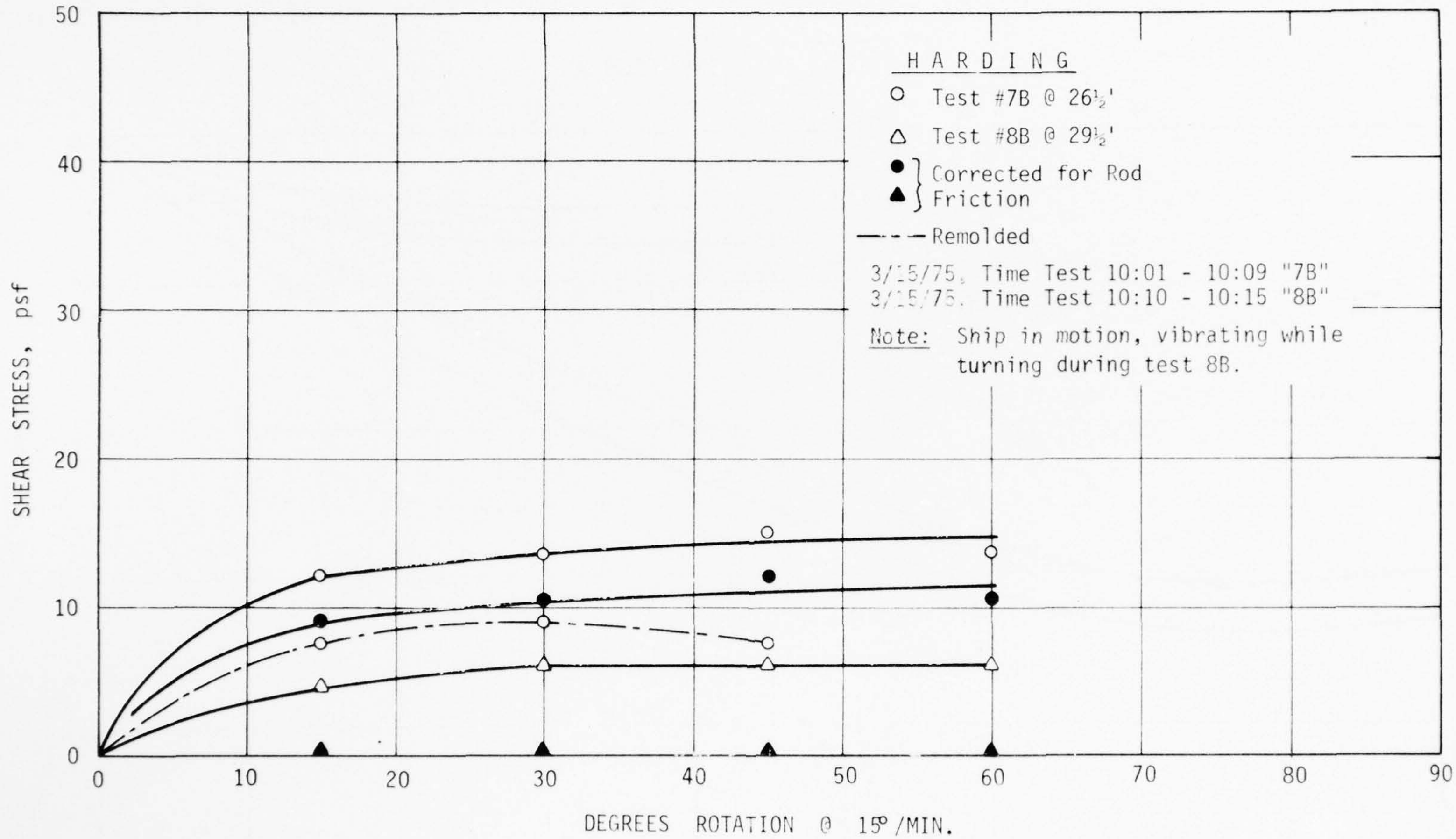


Figure B-10. Field Vane Shear Tests - HARDING

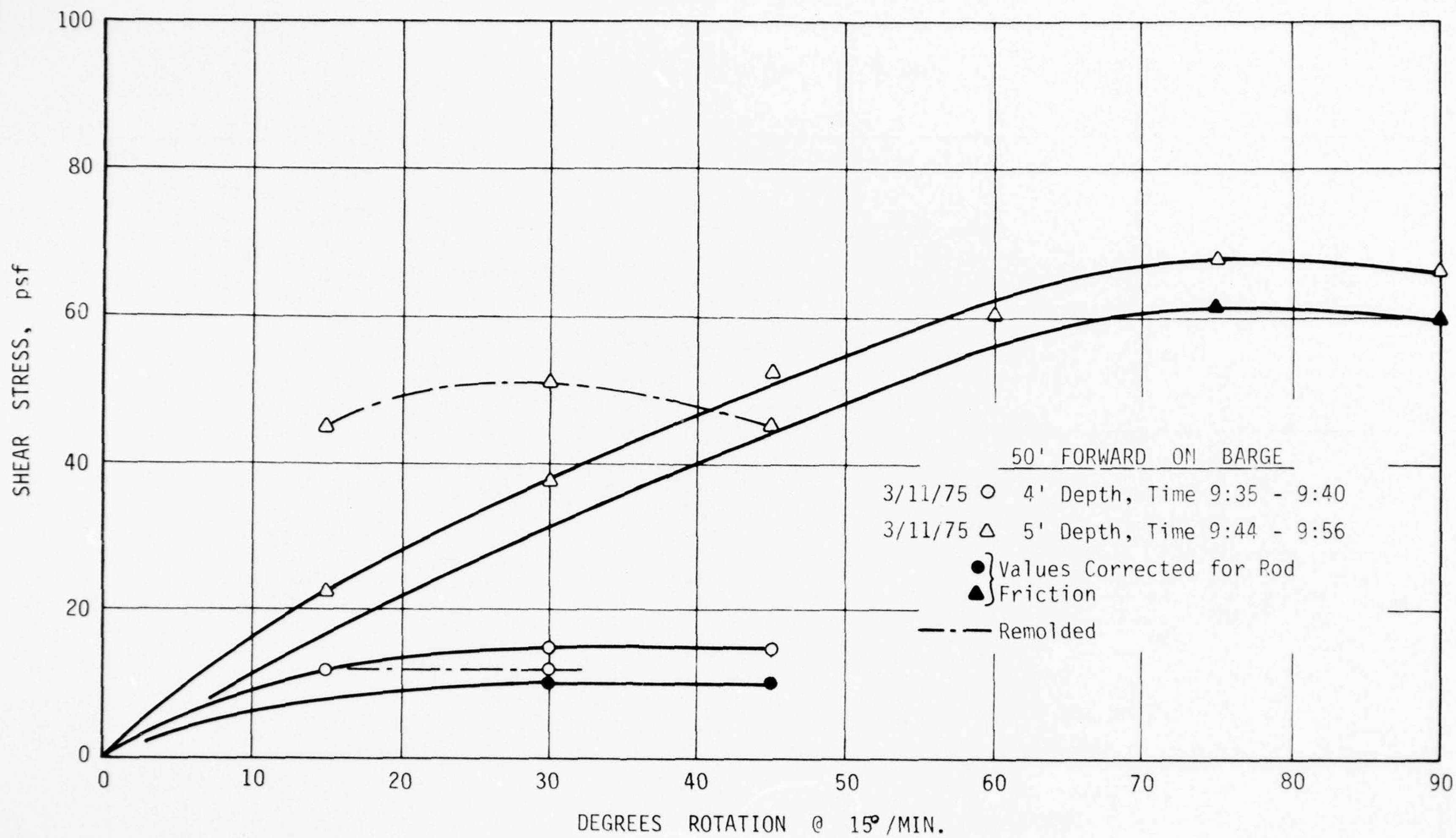


Figure B-11. Field Vane Shear Tests - BARGE

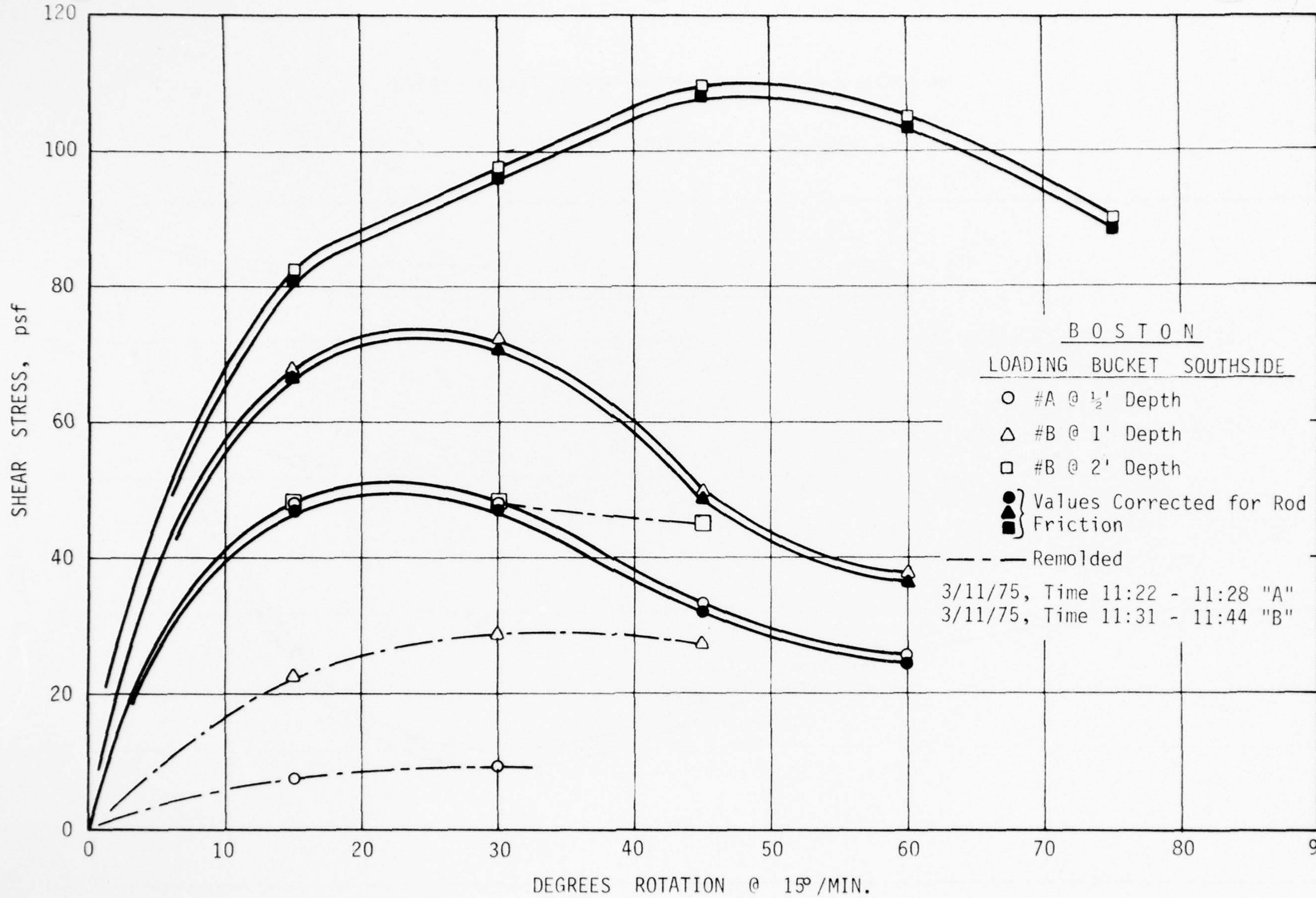


Figure B-12. Field Vane Shear Tests - BOSTON

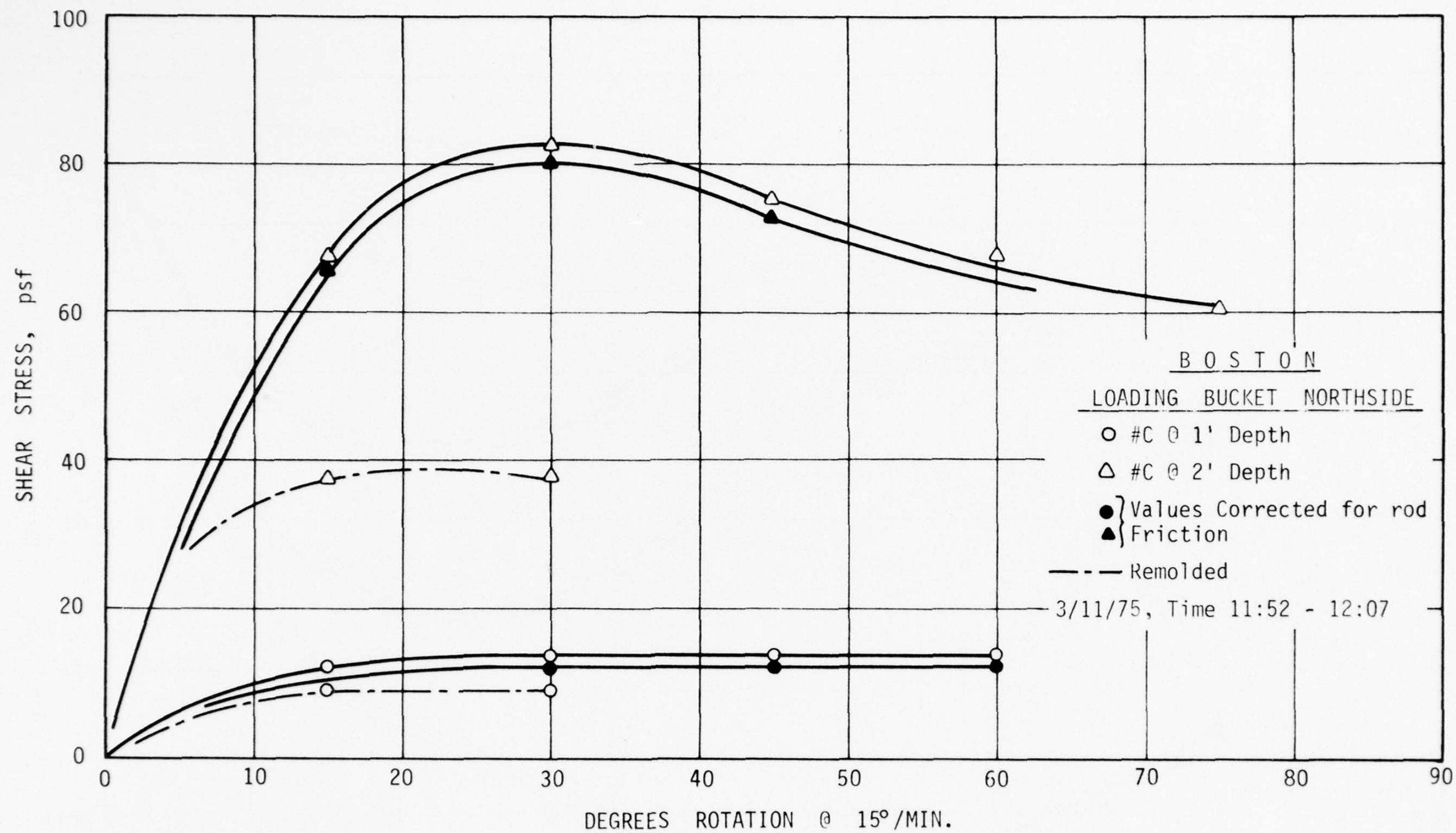


Figure B-13. Field Vane Shear Tests - BOSTON

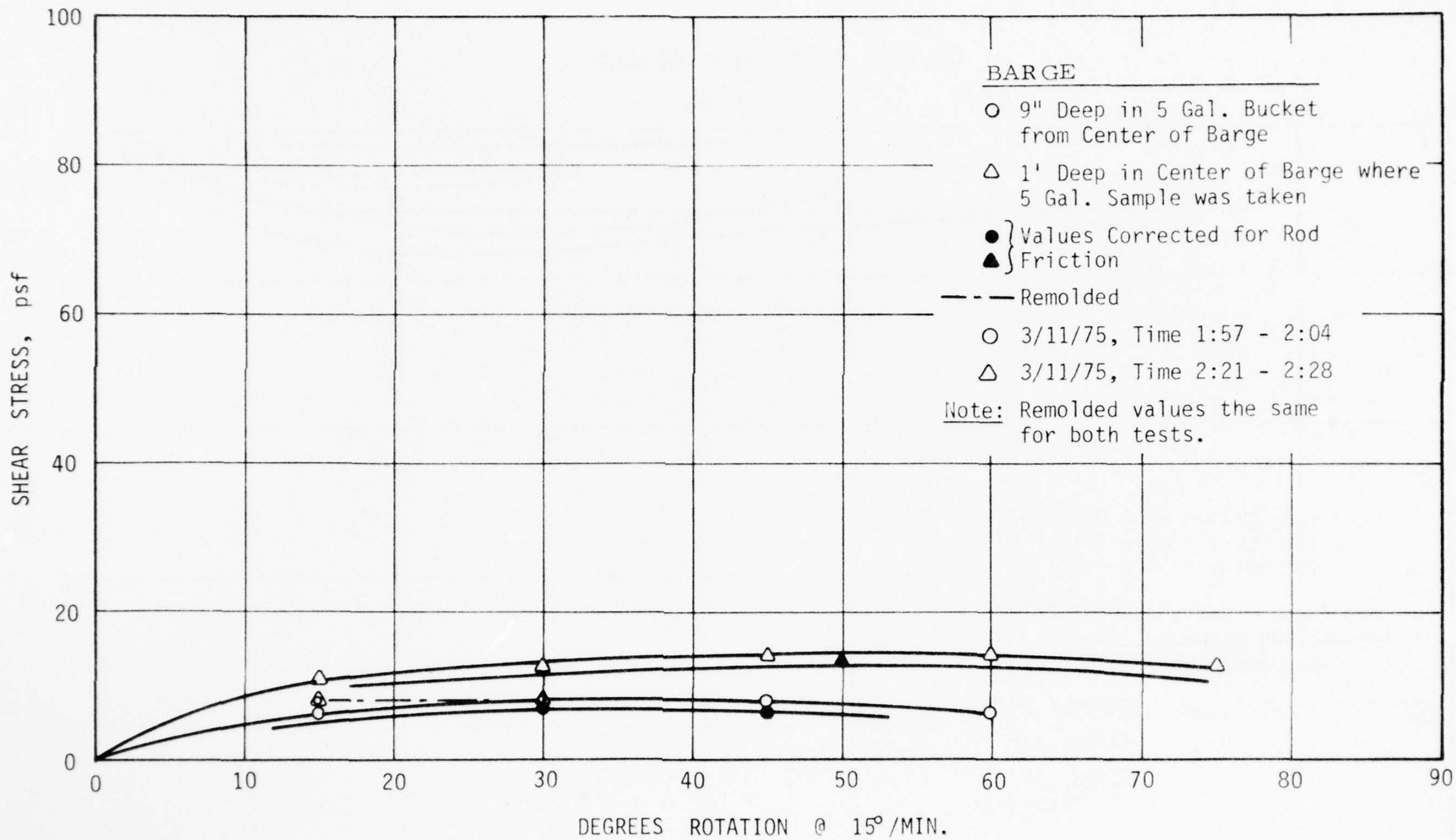


Figure B-14. Field Vane Shear Tests - BARGE

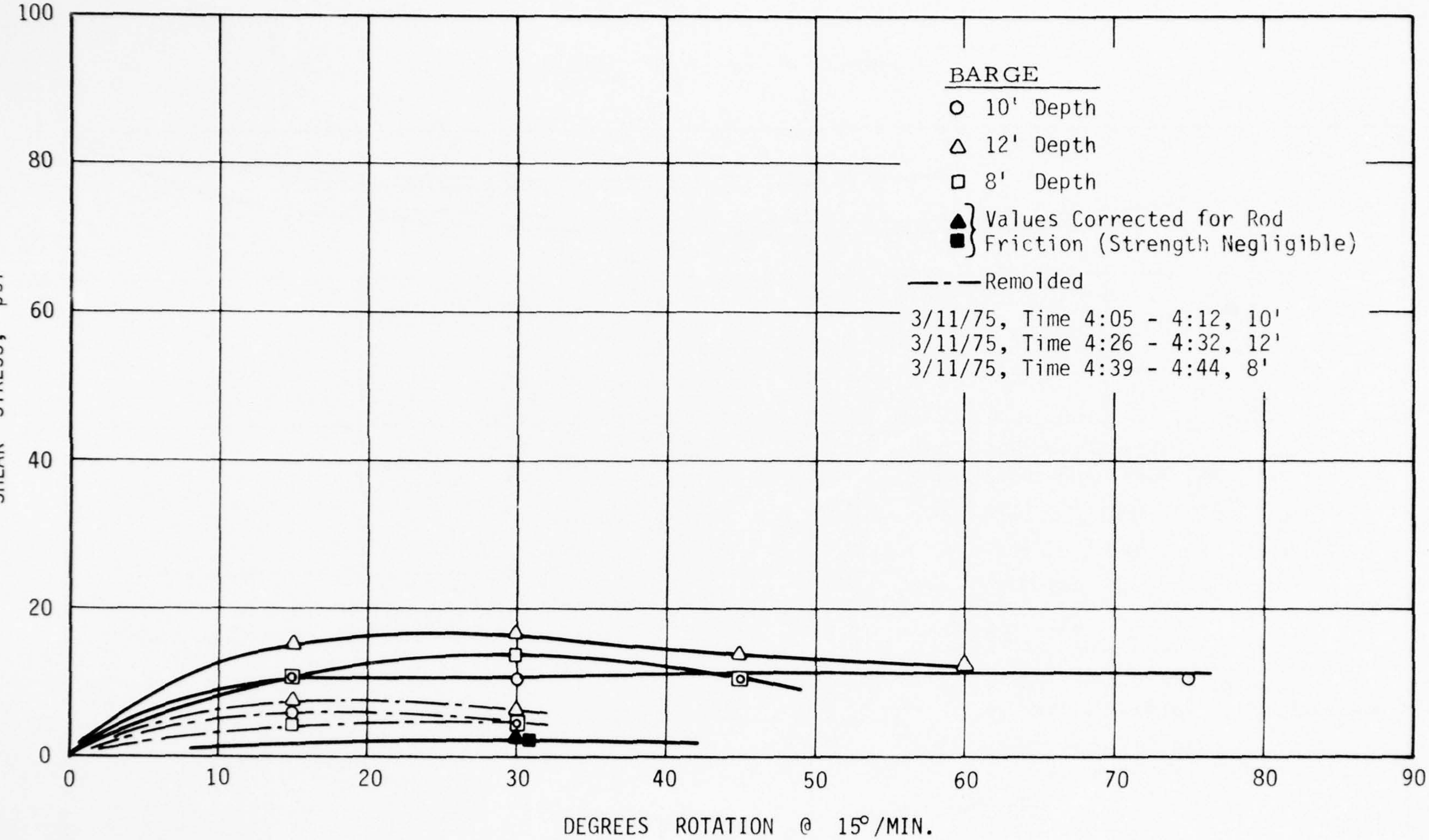


Figure B-15. Field Vane Shear Tests - BARGE

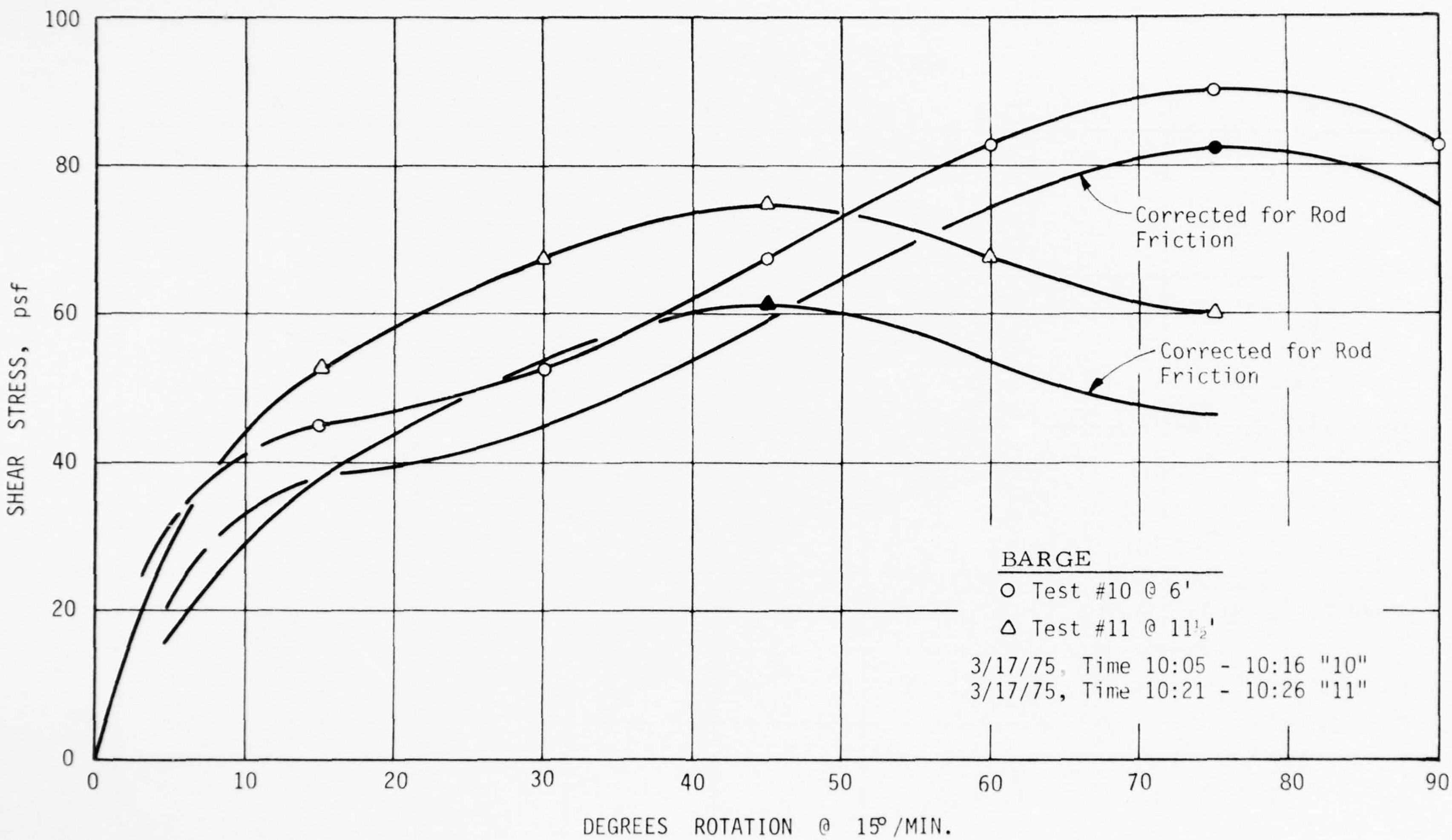


Figure B-16. Field Vane Shear Tests - BARGE

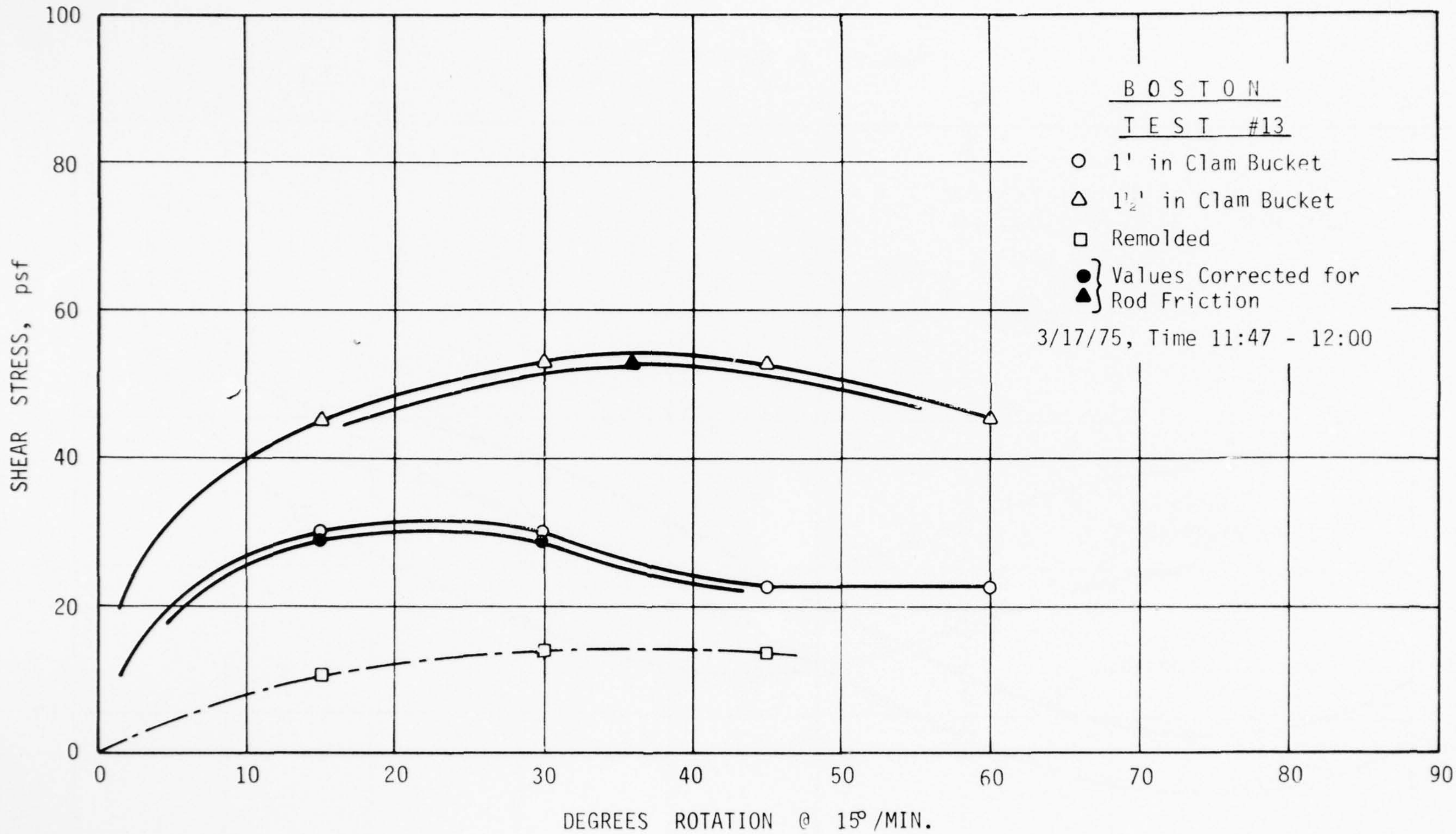


Figure B-17. Field Vane Shear Tests - BOSTON

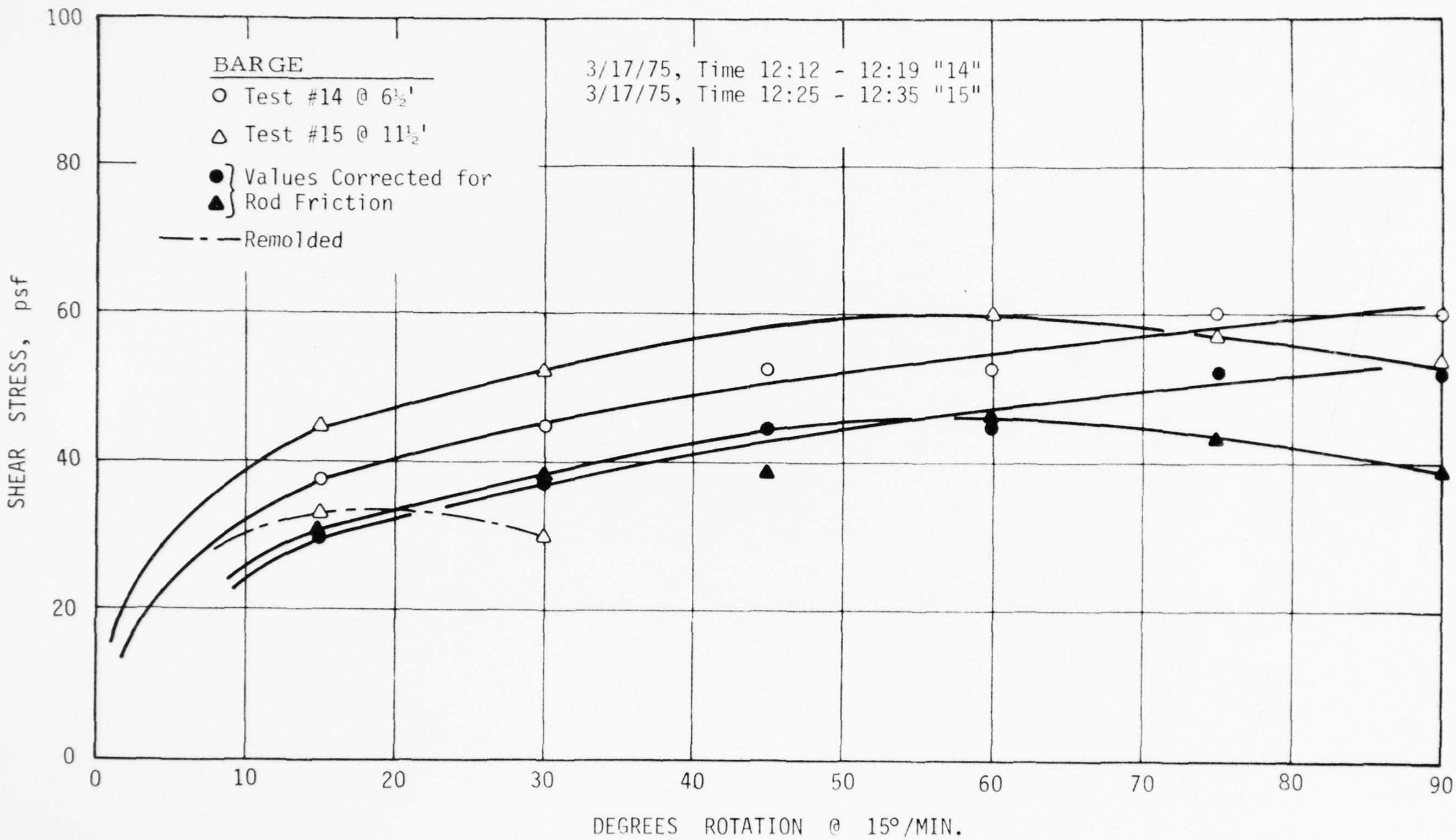


Figure B-18. Field Vane Shear Tests - BARGE

## APPENDIX C

## Measurement of Vibrations in Dredged Material

Most shipboard vibration measurements are made on bulkheads, decks, or other rigid surfaces. In addition to these conventional measurements, it was desired to make measurements directly in the dredged material and this required a special transducer.

To measure the harmonic motion of the water and sediment a probe containing harmonic velocity pickups and accelerometers was introduced into the fluid. The probe was supported by a 1 inch IPS pipe clamped to the upper structure of the dredge at about the mid-length of the forward section of hopper 3 and at about the third point on the width on the starboard side. The harmonic velocities and accelerations on the starboard hopper side adjacent to the probe were also measured.

An ideal probe would have the same density as the fluid being measured. However because of requirements for water tightness, threads to carry the transducers, and a short time frame for design, the probe built was considerably heavier than the water that it displaced. The following development analyzes the influence of this excess weight and the influence of the probe support on the measurements of the dredged material harmonic motion.

Consider a probe supported in a tank of fluid of density $\rho_f$ moving in simple harmonic motion, with an amplitude $A_o$, as shown in Figure C-1.

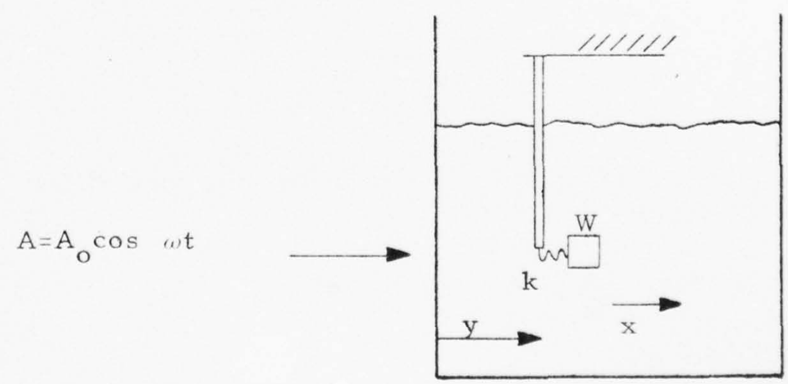


Figure C-1. Probe Model

Let:

| | | |
|---|---|---|
| V | = | probe volume |
| $\rho$ m | = | probe average volume |
| W | = | probe average density |
| k | = | probe support stiffness |
| x | = | probe motion |

Since the fluid is oscillating in simple harmonic motion, ($A=A_o \cos \omega t$) it will have an acceleration equal to $-A_o \omega^2 \cos \omega t$, and a corresponding pressure gradient, equal to $\frac{\delta P}{\delta y} = \left|\frac{\rho f}{g}\right| |accel.|$ . Under this pressure gradient the probe will experience a buoyancy force $= |V\rho f| \left|\frac{accel}{g}\right|$
In addition there will be impulsive forces, drag forces, etc. whose magnitude are determined by the relative motion between the probe and the fluid. These forces may be represented as F (x-A).

The forces acting on the probe are:

A buoyancy force, $+ |V\rho f| \left[\frac{\ddot{A}}{g}\right]$

A support force, $-kx$

A surface force, $-F(x - A)$

The equation of motion of the probe becomes:

$$V\rho f \frac{\ddot{A}}{g} - kx - F(x - A) = \frac{W}{g} \ddot{x}$$

In integrating this equation first consider the case where F(x-A) is neglected. Then:

$$\frac{W}{g} \omega^2 x - kx = \frac{V\rho f}{g} \omega^2 A_o \cos \omega t$$

and,

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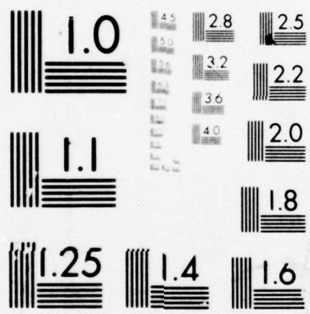


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$$x = \frac{\dfrac{V\rho f}{W} \quad A_o \cos \omega t}{\left(1 - \dfrac{kg}{W\omega^2}\right)}$$

It can be shown later that for most probe locations $\frac{kg}{W} = \omega n^2$ is very small compared to $\omega^2$. By setting $W = V\rho_m$, then $x = \frac{\rho f}{\rho m} A$. For the case where the weight of the probe is 25 pounds, the probe volume is 236 cu in, then the probe density is 0.1059 lb/cu in, the density ratio is $\frac{0.1059}{0.0361} = 2.934$, and, since drag forces were neglected, the probe would indicate vibrations 0.341 times those of the surrounding water.

Returning to the differential equation of motion, the addition of the $F(x - A)$ term will act to bring the measured probe vibration between 0.341 and 1 times the vibration in the fluid.

There appears to be very little information available that relates the resistance of a solid body to the harmonic motion between it and the surrounding water. Sarpkaya and Garrison [C-1] in a paper entitled "Vortex Formation and Resistance in Unsteady Flow" consider the response of a long circular cylinder to harmonic flow, generally of an amplitude larger than the cylinder diameter. From theoretical considerations of vortex formation, they show that the forces on the cylinder are the sum of a force related to the square of the fluid velocity plus a force related to the acceleration of the fluid. They then show that for small harmonic motions the inertia forces dominate and correspond to the forces generated by accelerating the entrained mass. The entrained mass is not easily determined. Patton [C-2] gives experimentally determined values for $M_h$ for rectangular parallelopipeds. Using his constant:

$$M_h = (2.32)(0.036)(5.625)^3 = 14.86 \text{ lb}$$

With this addition the differential equation of motion becomes:

$$V\rho_f \frac{\ddot{A}}{g} - kx - \frac{M_h}{g}(\ddot{x} - \ddot{A}) = \frac{W}{g}\ddot{x}$$

$$\left(\frac{V\rho_f}{g} + \frac{M_h}{g}\right)\ddot{A} - kx = \left(\frac{W + M_h}{g}\right)\ddot{x}$$

$$x\left[k - \frac{(W + M_h)}{g}\omega^2\right] = -\left(\frac{V\rho_f + M_h}{g}\right)\omega^2 A$$

and,

$$X = \left[\frac{V\rho_f + M_h}{W + M_h}\right]\left[\frac{A}{1 - \frac{kg}{(W + M_h)\,\omega^2}}\right]$$

The probe was carried by a 1 inch IPS pipe clamped to the ship structure at two locations, as shown in Figure D-2.

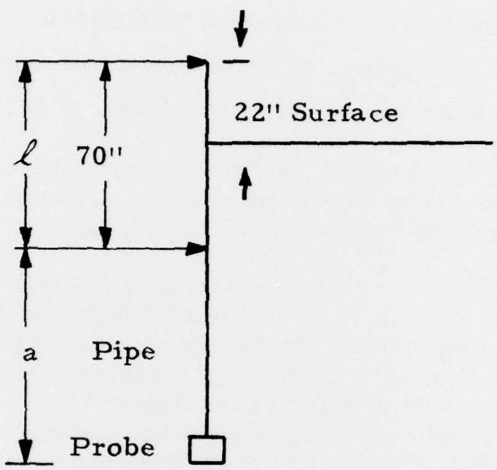


Figure C-2. Pipe support points

The support stiffness at the probe was:

$$k = \frac{3EI}{a^2(a + \ell)}$$

The natural frequencies are

$$\text{in air} \quad f_n = \frac{1}{2\pi}\sqrt{\frac{kg}{W}} = \sqrt{\frac{(386)\ k}{(4\pi^2)\ (25)}}$$

$$= \sqrt{0.3911k}$$

$$\text{in water} \quad f_n = \sqrt{\frac{(386)(k)}{(4\pi^2)\ (39.9)}} = \sqrt{0.2451k}$$

The probe dynamics are:

| Depth Below Surface | a | e + a | k lb/in | $f_n$ in air, cps | $f_n$ in water, cps | $\frac{x}{A}$ @ 20 cps |
|---|---|---|---|---|---|---|
| 5'-4" = 64 | 16 | 86 | 357.3 | 11.82 | 9.358 | 0.752 |
| 12'-6" = 150 | 102 | 172 | 4.396 | 1.311 | 1.038 | 0.589 |
| 17'-6" = 210 | 162 | 232 | 1.292 | 0.711 | 0.563 | 0.587 |
| 25'-0" = 300 | 252 | 322 | 0.385 | 0.388 | 0.307 | 0.587 |

## References


C-1. Sarpkaya and Garrison, "Vortex Formation and Resistance in Unsteady Flow," ASME J. of Appl. Mech., March 1963, pp. 16-23

C-2. Patton, ASME 65. WA/UNTZ




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